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PUBLIC ROADS
A JOURNAL OF HIGHWAY RESEARCH
UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

INDEX
TO VOLUME 6

MARCH 1925 - FEBRUARY 1926

ENTRIES APPEAR IN THIS INDEX

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A. E. S. C.—American Engineering Standards Committee.
fn.—footnote.
q.—quoted.
r.—reference, referred to.
r. to p.—reference(s) to publication(s).
U. S. D. A.—United States Department of Agriculture.
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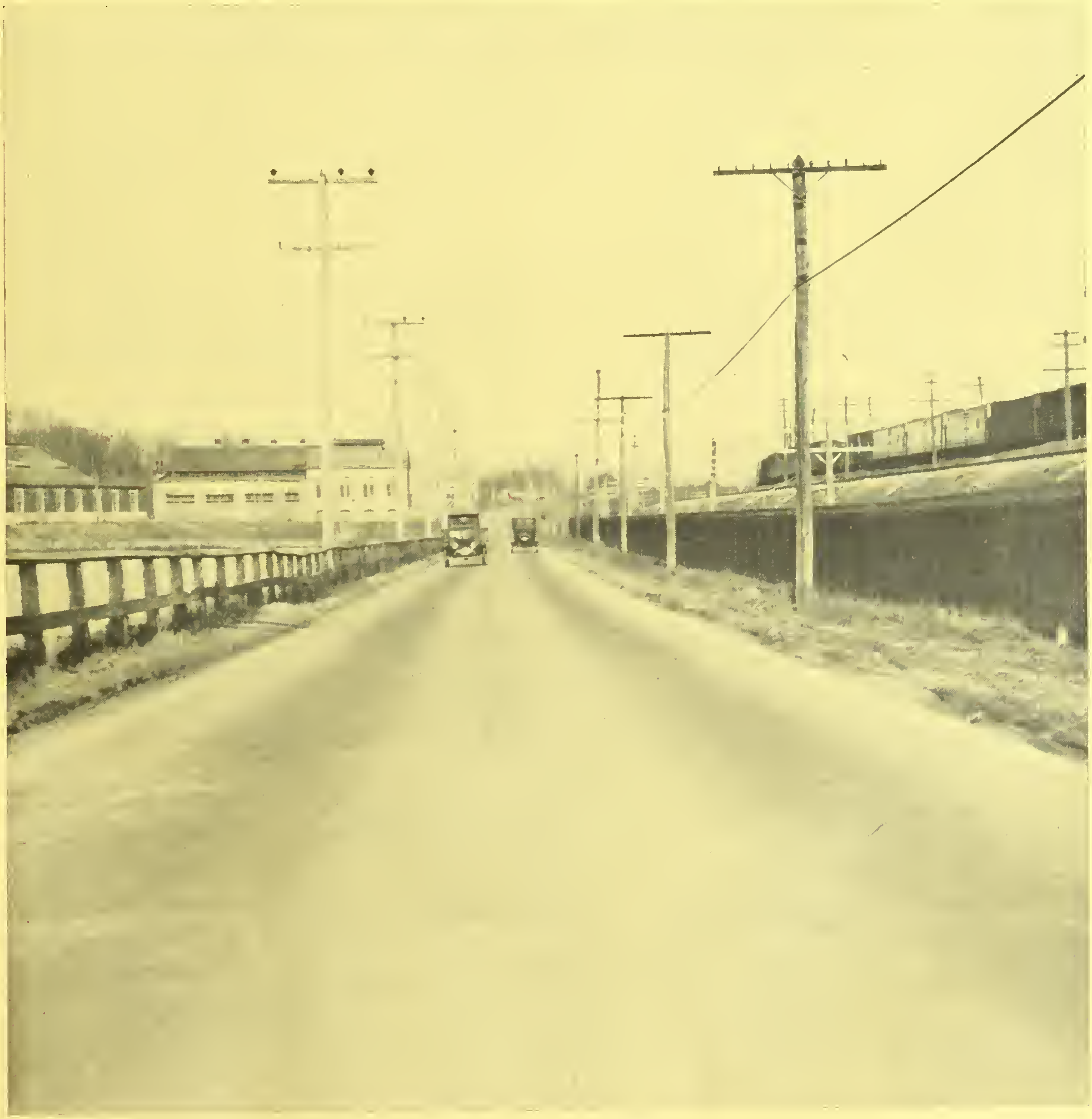

OIL STAINS ON A CONCRETE ROAD WHICH INDICATE THE LINES OF MAXIMUM TRAFFIC

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1925

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TRANSVERSE DISTRIBUTION OF MOTOR VEHICLE TRAFFIC ON PAVED HIGHWAYS

By J. T. PAULS, Associate Highways Engineer, United States Bureau of Public Roads

A LIMITED investigation conducted recently by the Bureau of Public Roads for the purpose of determining the transverse distribution of traffic on paved highways, has developed a number of very significant facts with respect to the effect of road width, curves, shoulder conditions, grades, crown of surface and other physical factors on the distribution of the traffic laterally. Although the investigation has not been sufficiently extensive to develop relations definite enough to describe as the laws of lateral distribution, the observations made are highly interesting and of immediate practical value, with respect to highway design and traffic regulation. Previous traffic surveys [1] have given the net and segregated tonnages and the number of applications of the load per unit of time, but so far as is known, this is the first attempt to establish the transverse distribution of traffic on pavements.

LOCATION AND CHARACTER OF INVESTIGATION

The observations were made in the vicinity of Washington, D. C., and locations were selected which were considered to be representative of the several conditions it was desired to study. At each point selected, the paved width was marked with paint into 1-foot sections. The right rear wheel passages in each direction were noted by different recorders with tally registers. The known distance between the wheels was utilized to add to the register the left wheel passages. The observations represent the total rear wheel passages considered as the sum of the traffic in both directions.

The truck and passenger traffic were registered separately and each considered as 100 per cent in plotting the graphs. Trucks of less than 2 tons capacity were not counted with the heavier types; because it was believed that such trucks would be equivalent to passenger cars.

The quantitative results obtained from the investigation are shown on the accompanying charts. The percentage of traffic passing over any 1 foot of width is given in terms of the total rear-wheel traffic in both directions, considered as 100 per cent. The average position of the outer wheels, as indicated on the charts, has been determined by locating the center of gravity of the area representing the outer wheel passages only. The corresponding average position of the inner wheels was then located at the known gauge distance from the outer wheels. The distribution of the traffic moving in the two directions is represented by the ordinates of the shaded and unshaded areas. The sum of the total ordinates at the 1-foot divisions represents the total traffic moving in both directions considered as 100 per cent. The average net clearance between vehicles moving in opposite directions has been calculated from the computed average location of the outer wheels on the basis of what appears to be the average overall width of vehicles. For passenger cars this overall width is taken at 5.7 feet, which is practically a constant for all makes of cars. For trucks a width of 7.3 feet has been used. If the maximum legal width of 8 feet obtaining in a number of States were used, the average truck clearances shown on the charts would be reduced accordingly. Figure 1 shows the dimensions used. The highway conditions at each of the points of observation are shown by the photographs accompanying the charts; and are tabulated and the results further described in Table 1.

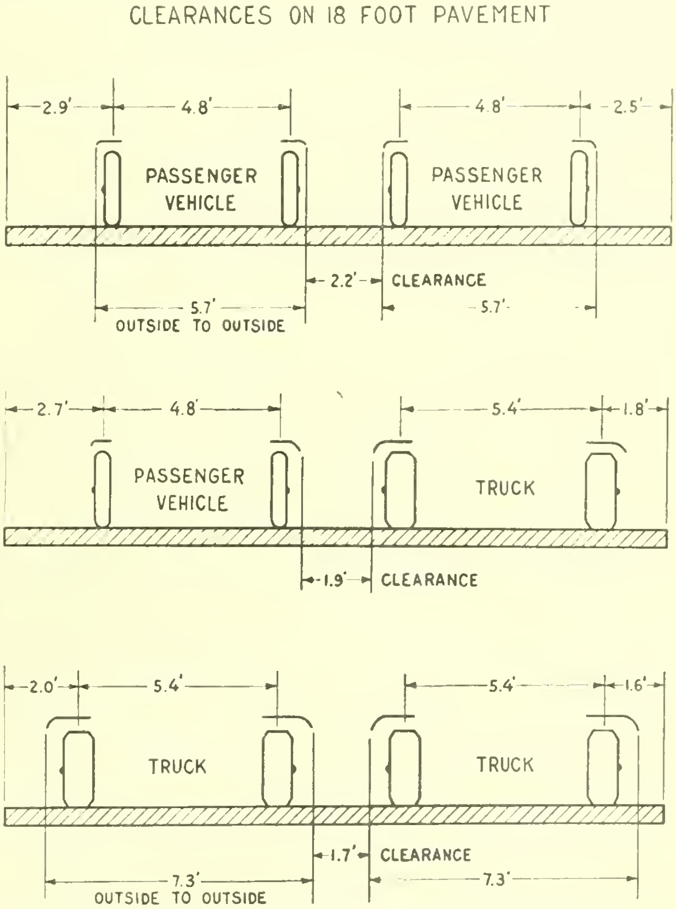

FIG. 1.—Dimensions of passenger vehicles and motor trucks assumed for the purpose of computing average clearance; and clearances on an 18-foot pavement under three conditions of traffic (based on data from test 4. Fig. 5)

SUMMARY OF FINDINGS

The principal results of the observations with respect to the distribution of traffic and the effect of various physical conditions are as follows:

Pavement width.—It seems to be definitely established that 18 feet is the minimum pavement width which will permit passenger vehicles and trucks driven in the preferential position to pass in safety and with a reasonable amount of clearance. Trucks have almost as much clearance as passenger cars, because the former are habitually driven about 1 foot closer

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[1] See Public Roads, Vol. 5, No. 1. March, 1924. Connecticut Highway Transportation Survey, p. 14, and Vol. 5, No. 10, December, 1924. Highway Traffic in California, p. 12.

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TABLE 1.—*Data showing conditions on various pavements and distribution of motor-vehicle traffic*

Test No.	Type of vehicle	Pavement type	Crown	Super-elevation	Average speed	Condition of shoulders		Other features		Width of pavement	Traffic line	Grade		Horizontal curve			Average distance of wheel from edge		Average clearance	Maximum traffic from one wheel		
						Left	Right	Left	Right			Left	Right	Degree	Length	Direction right side	Left	Right		Left side	Right side	On pavement
			In.	*In.*	*Miles per hr.*					*Ft.*		*Per cent*	*Per cent*	° ′	*Feet*		*Ft.*	*Ft.*	*Ft.*	*Per cent*	*Per cent*	*Per cent*
1	Passenger	Concrete	2¼		25	Bad	Bad			18							3.5	3.2	0.9	8.1	8.4	11.8
4	do	do	2¼		25	Fair	Fair			18							3.3	2.5	2.2	8.6	10.1	10.9
4	Truck	do	2¼		12	do	do			18						¾	2.0	1.6	1.6	12.7	8.9	12.7
23	Passenger	Bituminous macadam.	4		25	do	do			22							3.7	3.7	4.2	5.1	6.8	8.1
23	Truck	do	4		12	do	do			22							2.8	2.5	3.9	11.1	4.4	11.1
17	Passenger	Granite blocks.	3		18			Curb	Curb	24							3.3	4.2	6.1	6.7	5.6	7.0
17	Truck	do	3		10			do	do	24							2.3	3.8	5.1	10.2	8.5	10.2
9	Passenger	Bituminous macadam.	3½		25	Fair	Fair			14		+2	−2				1.6	2.2	−0.2	10.0	9.7	19.5
7	do	do	7½		22	do	do	Ragged edge	Ragged edge	15		+2	−2				2.2	3.6	−1.2	9.1	8.5	17.4
7	Truck	do	7½		12	do	do	do	do	15		+2	−2				1.7	2.8	−2.3	11.2	9.5	20.7
8	Passenger	do	6		20	do	do	do	do	16		+6	−6				3.0	3.3	−0.7	7.0	13.2	18.2
5	do	do	3¾		25	Good	do	(1)	(1)	20		−1	+1				4.5	3.2	1.9	8.6	5.5	9.6
5	do	do	3¾		30	do	do	(2)	(2)	20		−1	+1				3.6	2.1	3.9	7.2	11.0	11.7
5	Truck	do	3¾		15	do	do	Ragged edge	Ragged edge	20		−1	+1				1.8	1.2	4.2	10.4	11.0	11.7
20	Passenger	Concrete	2¼		15	Fair	do			18		+7	−7				3.0	3.3	1.3	7.2	7.2	12.5
10	do	Bituminous macadam.	3¾		20	Gutter	Gutter	Rutting	Rutting	20		−6	+6				4.1	4.7	0.8	6.6	7.2	10.3
10	Truck	do	3¾		10	do	do	do	do	20		−6	+6				3.6	3.1	0.5	10.6	7.7	11.6
12	Passenger	do	6		20	do	do	(3)	(3)	24	Center						4.4	4.2	5.0	11.6	6.4	11.6
12	Truck	do	6		12	do	do	Rutting	Rutting	24	do						4.0	4.7	2.5	10.0	8.8	10.0
11	Passenger	Concrete	2¼		20	Good	Good			18	do	−1	+1				2.6	2.6	2.4	10.8	10.8	11.9
6	do	do	2½		25	Fair	Fair			20	do	−2	+2				3.4	3.1	3.1	10.3	7.7	10.7
6	Truck	do	2½		12	do	do			20	do	−2	+2				2.7	2.4	2.1	10.8	15.8	15.8
16	Passenger	Bituminous macadam.		9	20	do	Gutter	(4)	(4)	24	do			18	312	Inner	5.0	3.0	5.6	7.0	7.7	7.7
16	Truck	do		9	12	do	do	(4)	(4)	24	do			18	312	do	4.5	2.5	4.2	7.6	8.6	8.6
19	Passenger	do		6	18	Gutter	Fair			24	do			20	175	do	3.6	1.7	8.3	8.8	5.6	9.6
13	do	do		5	25	Fair	do			20				15	100	do	4.7	2.2	2.7	6.3	9.2	10.3
13	Truck	do		5	12	do	do			20				15	100	do	2.6	1.4	3.2	9.0	16.7	16.7
14	Passenger	do		5	25	do	do	(2)	(2)	20				8	400	Outer	2.4	4.4	2.8	11.2	5.1	12.1
14	Truck	do		5	12	do	do	(2)	(2)	20				8	400	do	1.2	3.7	2.3	15.2	6.9	15.2
18	Passenger	do		8	25	do	do			22				11 30	230	Inner	6.0	3.2	2.4	5.4	4.8	10.2
18	Truck	do		8	12	do	do			22				11 30	230	do	5.8	1.9	1.5	4.0	18.0	18.0
22	Passenger	Concrete		5	20	do			Fill and guard rail.	18		+2	−2	25	375	Outer	2.2	5.8	−0.4	9.8	4.7	13.7
21	do	Bituminous macadam.	5½		22	do	Fair			22		+7	−7				4.2	5.1	2.3	6.4	6.7	12.4
21	Truck	do	5½		12	do	do			22		+7	−7				1.7	2.6	4.9	12.3	12.3	13.1
3	Passenger	Concrete	2¼		25	Bad	Bad			18							3.8	4.0	−0.2	9.5	8.5	15.5

[1] Taken at night.

[2] 2½-foot concrete edges.

[3] Bridge abutment.

[4] View obstructed by bank at right.

to the edge of the pavement than the latter. It is interesting to note that drivers of vehicles apparently will sacrifice clearance rather than drive closer to the edge than they instinctively feel to be safe.

In suggesting a minimum width of 18 feet the observations bear out current practice based upon gross observation. A width of 20 feet would apparently permit the passing of vehicles with greater ease and safety and without excessive cost. It is apparent, however, that a width in excess of 20 feet is unnecessary for a two-way pavement on tangents either for trucks or passenger vehicles, since the clearance between the vehicles at their preferred driving position is sufficient on a pavement of this width to permit passing with safety.

That the used width of a pavement may frequently be considerably less than its apparent width is strikingly brought out by several of the observations. In several instances it is shown that the outer foot of the surfaced section is totally ineffective because of a bad shoulder condition, a closely set guardrail, a steep crown, a bad gutter or the ragged edge of a bituminous macadam surface; and in one case a 24-foot pavement has been found to be effectively not more than 20 feet wide because of the abutments of an overhead railroad bridge which are crowded close to the edge of the pavement.

Curves.—The observations provide specific support for the prevailing belief that traffic in general shifts toward the inside edge of the pavement in rounding horizontal curves. At variance with the usual assumption, however, the trucks are found to shift their course toward the inside of the curve less than the passenger vehicles; and under all circumstances the truck drivers are found to adhere more closely to the edges of the pavement than the operators of passenger vehicles.

It is found that traffic moving on the outside of the curve shifts its course farther in the direction of the inside than the traffic moving in the opposite direction, which is limited in its choice of a course by the proximity of the pavement edge. It follows, therefore, that unless the pavement is widened on the curves the normal tangent clearance between the two lines of vehicles will be reduced. The behavior of the traffic on the curves is influenced, however, by the presence or absence of traffic center lines; the amount of superelevation and whether the superelevated section has a plane or crowned surface; the condition of the shoulders and roadsides; the presence or absence of roadside structures such as guardrails, culvert head walls, etc., either on the inside or outside of the curve, and the degree of curvature and tangent width of the pavement. The observations shows that traffic center lines do accomplish the separation of the traffic; and they apparently show that a crowned section, even though superelevated, has the effect of causing the traffic to seek the favorable slope on the inside of the curve. But because of the presence of complicating factors the relation between the degree of curvature and the tangent width of the pavement and the amount of widening required on the curve is not revealed.

Grades.—The distribution of traffic on grades seems to differ only slightly from the distribution on level roads. There is, however, a noticeable departure from the normally preferred course by the traffic moving down hill on light grades. In practically every case observed the traffic under this condition moves slightly toward the center of the road. The explanation advanced is that light down grades do not suggest reduction of speed, but that the drivers do take the precaution of moving slightly away from the edge of the pavement. No such tendency is observed on heavy grades and the presumption is that operators reduce speed on such grades and, at the reduced speed, the instinctive fear of the pavement's edge is lessened.

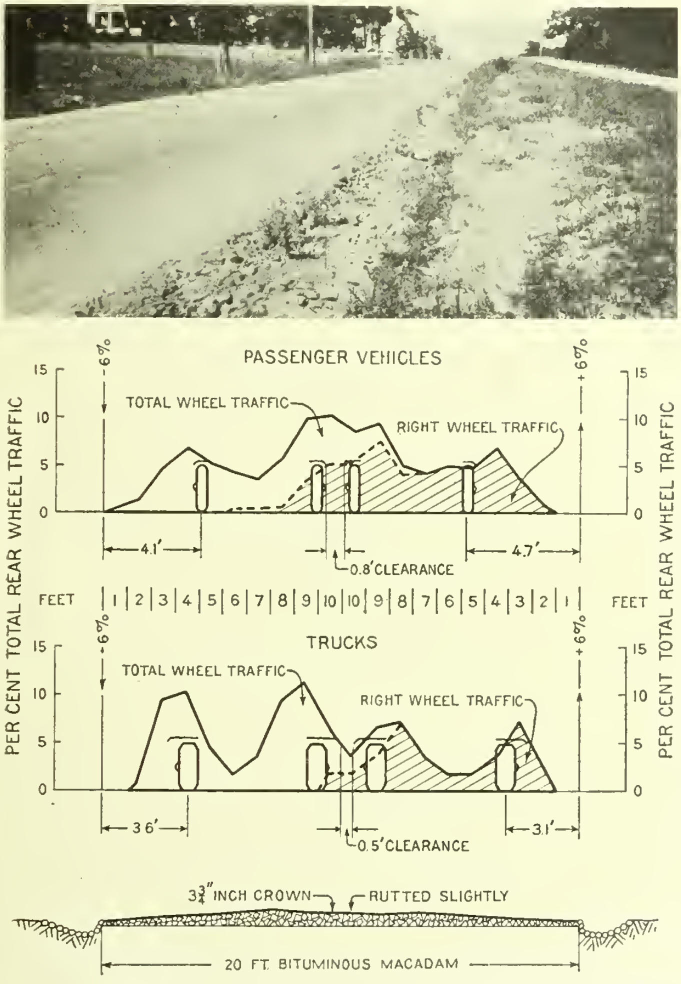

FIG. 2.—Test 10, made on a 20-foot bituminous macadam road with a bad cobble gutter at each side, shows the effect of the concentration of traffic at the center of the surfaced way caused by the condition of the gutter. Increasing concentration at the center may be expected to follow the development of the ruts, leading to still more rapid deterioration of the surface

Other factors affecting lateral distribution.—The observations indicate that other factors, such as the crown of the road, the condition of the shoulders, the presence of concrete shoulders at the sides of a bituminous pavement, center traffic lines, and the presence of roadside structures such as guardrails, culvert headwalls, etc., have a noticeable effect upon the lateral distribution of the traffic. Excessive crown is shown to cause a concentration of traffic at the center of the roadway. A bad shoulder has a similar effect, and guardrails and other similar structures suggesting roadside danger also cause the traffic to move away from the apparent danger and toward the center of the road. Smooth, white concrete shoulders at the edge of a black surface seem to lure the traffic toward the side, and center lines, on tangents as well as on curves, exert a marked separatory influence.

In one case roughness of the surface at a distance from the edge approximately equal to the wheel gauge of passenger vehicles appears to have reduced the used width of the pavement by 1 foot, because of the reluctance of operators to drive with their inner wheels on the rough surface.

Maximum traffic concentrations.—Considering all widths of pavement and all conditions, the maximum percentage of the total traffic concentrating upon a single foot of width is found at the center of a 14-foot pavement for passenger vehicles and at the center of a 15-foot roadway for trucks. The heaviest concentration on a pavement 18 feet wide or wider is shown to be approximately half of these maxima for both classes of vehicles.

On curves and on light grades the point of greatest concentration shifts from the center toward the inside of the curve and the uphill side of the road, respectively. On concrete roads the lines of maximum concentration are clearly marked by the dark stains caused by oil which drops from the center of the drip pans of the vehicles. (See illustration on cover.) The darkest part of these stains which, because of the favorable slope of the drip pans, are always more intense on the uphill side of the road, corresponds to the center of the track of greatest concentration. The heaviest wheel concentration may generally be expected at a distance from the center of the stain equal to one-half the wheel gauge of the vehicles toward the center of the road; and when the distances thus measured reach or slightly pass the center of the road a maximum concentration for the traffic may be expected at the center. Markings, in appearance similar to those caused by drip-pan droppings, may sometimes be observed on concrete roads near points where they abut bituminous surfaces. In such cases, however, the marks are caused by the wheels themselves, and indicate directly the lines of greatest wheel concentration. Similar lines develop on bituminous pavements shortly after the application of a seal coat, caused, in this case, by the earlier ironing down of the surface application at the lines of greatest traffic.

Bearing upon the design of pavements.—One of the most useful results of the observations is the evidence they present with regard to the points of application of the critical motor truck wheel loads. By demonstrating that motor trucks are habitually driven closer to the edge than passenger vehicles, and by showing that the average distance of the rear wheels of the trucks from the edge is little more than 1½ feet on most pavements, the study emphasizes the wisdom of the thickened-edge design, and indicates that the assumption of a point of application of the load at a distance of 6 inches from the edge is conservative.

The study shows convincingly that a census of total traffic may not always describe the full service rendered by a particular highway. In one of the observed cases, for example, 18 per cent of the total number of wheel applications came within a foot of the edge of the pave-

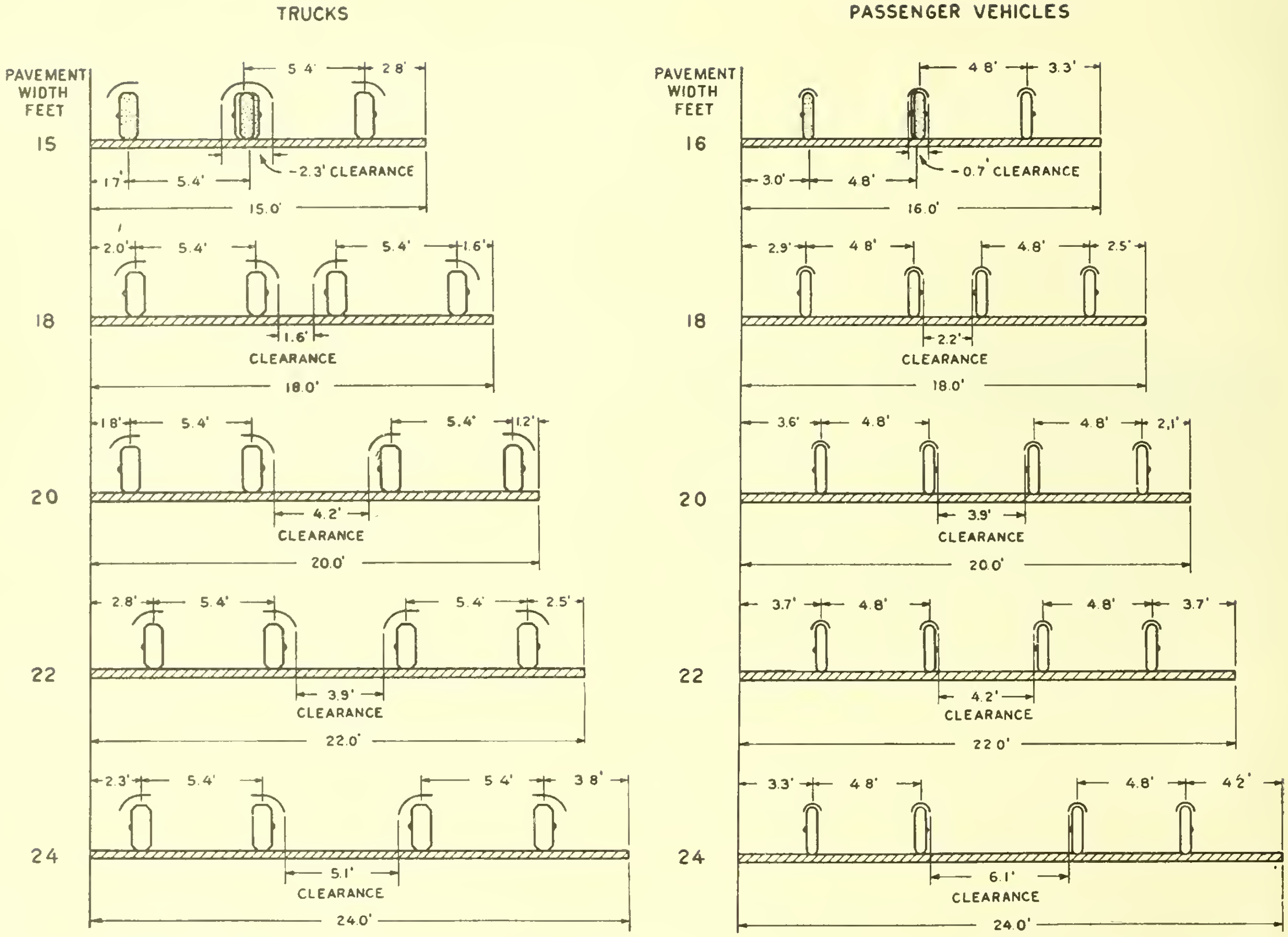

FIG. 3.—Average distribution of traffic on road surfaces of various widths from 15 to 24 feet. Eighteen feet is the minimum width which gives positive clearance for all vehicles in passing at the preferred position. The clearance increases with the width of the pavement, but becomes unnecessarily large when the width exceeds 20 feet. Widths of 22 and 24 feet are apparently excessive for two lines of traffic and not great enough for three lines

ment; in another case only 4 per cent of the total wheel passages were within the first foot. What effect such a difference may have upon the design of the pavement is a matter which will need to be considered carefully in the development of rational methods of road design.

Although the greater concentration of the traffic at one point than at another is likely to cause little difference in the surface wear by attrition, it is possible that a study of the distribution of traffic in connection with surface and subgrade failures may show a more intimate connection than at first glance may appear. One of the roads observed in this investigation may be taken as an example of such a failure. The road, observed in test 10 (Fig. 2), has a 20-foot pavement, but the presence of a bad cobble gutter at each side has forced the traffic to concentrate on the central portion of the pavement, resulting in the formation of ruts which cause a still higher concentration.

Although this paper presents no data relative to the effect of snow upon lateral distribution it may be pertinent to point to the experience of many of our cities during the past winter. The extensive damage revealed when finally the snow blanket of last January was removed—obviously the result, mainly, of the concentration of traffic in snow ruts—should convince the least foresighted of the falsity of that economy which withholds funds for snow removal only to expend greater sums for street repair.

WIDTH OF 18 TO 20 FEET AMPLE FOR TWO-WAY TRAFFIC

Assuming that the proper width of a two-way pavement is that width which will provide safe clearance between passing vehicles without forcing their drivers to reduce the distance between the outer wheel and the edge which is instinctively felt to be safe, it appears from the observations that a width of 18 feet is the minimum which should be adopted for two-way passenger-vehicle traffic and that widths greater than 20 feet are excessive. Trucks require 20 feet for safe passing, but do not fully utilize additional width.

Table 2 presents data on the habits of drivers operating over roads of various widths from 14 to 24 feet, under conditions of fair shoulders, level grades, and on tangents. From this table it appears that passenger-car drivers habitually maintain a distance of from 1½ to 4 feet between the outer wheel and the edge of the pavement, and that truck drivers operate somewhat closer to the edge, but prefer not to approach closer than 1½ feet. These distances, then, define the preferred tracks of the two kinds of vehicles, and may be taken as criteria in deciding upon the adequacy of any given road width. Viewed in this way it appears that pavements less than 18 feet in width are decidedly too narrow, since they provide no clearance for passenger cars or trucks. The 18-foot width is apparently great enough for passenger-car traffic but not quite wide

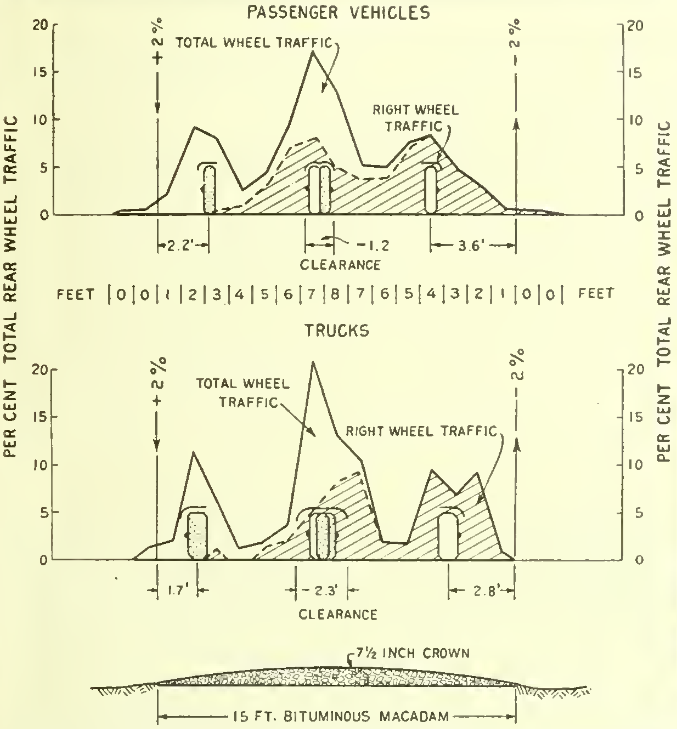

FIG. 4.—Test 7, made on a 15-foot road, shows how the maximum concentration of traffic occurs at the center on roads of such narrow width. The steep crown in this case accentuates the concentration at the center; and the effect of a slight downhill grade is noticeable in the greater distance of the average wheel position from the edge on the downhill side

enough for trucks. The width of 20 feet gives ample clearance for trucks and is apparently not excessive for automobiles. Further increase in width is apparently utilized by the drivers to increase the center clearance, but the added clearance is obviously not greatly needed for safety, for it is shown that the presence of a railroad bridge abutment near the sides of a 24-foot surface (test 12) will cause the truck traffic to give up 2.6 feet of clearance, and a slightly rough surface on a 22-foot road (test 21) reduces the clearance of passenger vehicles by 1.9 feet.

The motor-truck driver, frequently maligned by irate motorists as a road hog, finds unexpected comfort in the graphs which clearly show that he consistently drives closer to the edge of the pavement than his defamers. Before framing a retort, however, it would be well for him to remember that the graphs show him to be a consistent driver rather than a courteous one. The well-developed peaks representing his wheel tracks show that he adheres closely to a fixed path at his chosen distance from the edge, departing from it neither to occupy the center of the road when alone nor to give greater space to another vehicle in passing. The slower speed of his vehicle seems to permit him to drive closer

TABLE 2.—*Average traffic lanes and clearances of motor trucks and passenger vehicles on pavements of various widths. All on tangents with fair shoulders*

PASSENGER VEHICLES

Test No.	Pavement type	Width	Average distance of outer wheel from edge		Average clearance
			Left	Right	
		Feet	*Feet*	*Feet*	*Feet*
9	Bituminous macadam	14	1.6	2.2	−0.2
8	do	16	3.0	3.3	−0.7
4	Concrete	18	2.9	2.5	2.2
6	do	20	3.4	3.1	3.1
23	Bituminous macadam	22	3.7	3.7	4.2
21	do	22	4.2	5.1	[1] 2.3
17	Granite block	24	3.3	4.2	6.1
12	Bituminous macadam	24	4.4	4.2	[2] 5.0

MOTOR TRUCKS

Test No.	Pavement type	Width	Left	Right	Average clearance
7	Bituminous macadam	15	1.7	2.8	−2.3
4	Concrete	18	2.0	1.6	1.6
6	do	20	2.7	2.4	2.1
23	Bituminous macadam	22	2.8	2.5	3.9
21	do	22	1.7	2.6	[1] 4.9
17	Granite block	24	2.3	3.8	5.1
12	Bituminous macadam	24	4.0	4.7	[2] 2.5

[1] This road has 2½-foot concrete shoulders on each side, which normally would tend to increase the clearance. The presence of a rough place in the pavement about 6 feet from each edge causes passenger vehicles to forego the smooth concrete edge, because to do so would place their inner wheels on the rough place. Motor trucks are not affected, because their broader gauge permits them to use the concrete shoulder without running with their inner wheels on the rough place

[2] Railroad bridge abutments at each side of pavement cause reduction of clearance

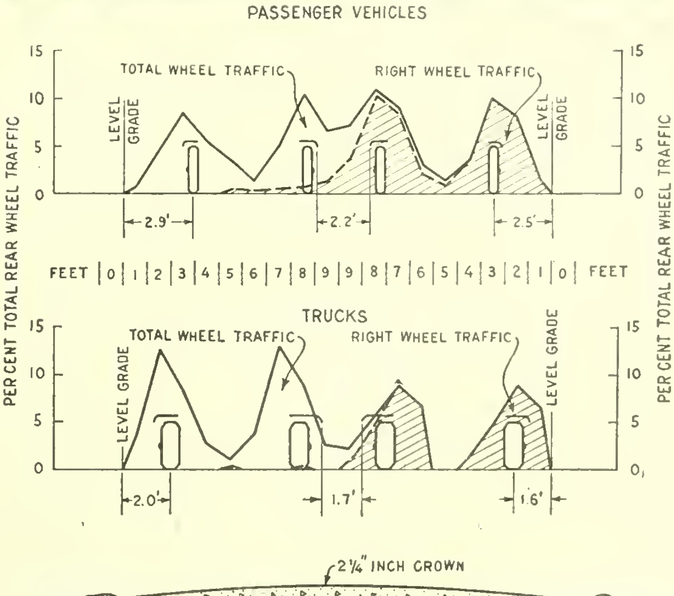

FIG. 5.—Test 4, on an 18-foot concrete pavement, shows how the center traffic concentration which characterizes the narrow roads is relieved by widening. The good shoulders on this road encourage traffic to use the surface to the very edge. Contrast this condition with that shown in Figures 15 and 16 where a bad shoulder condition clearly discourages the use of the edges of the surface

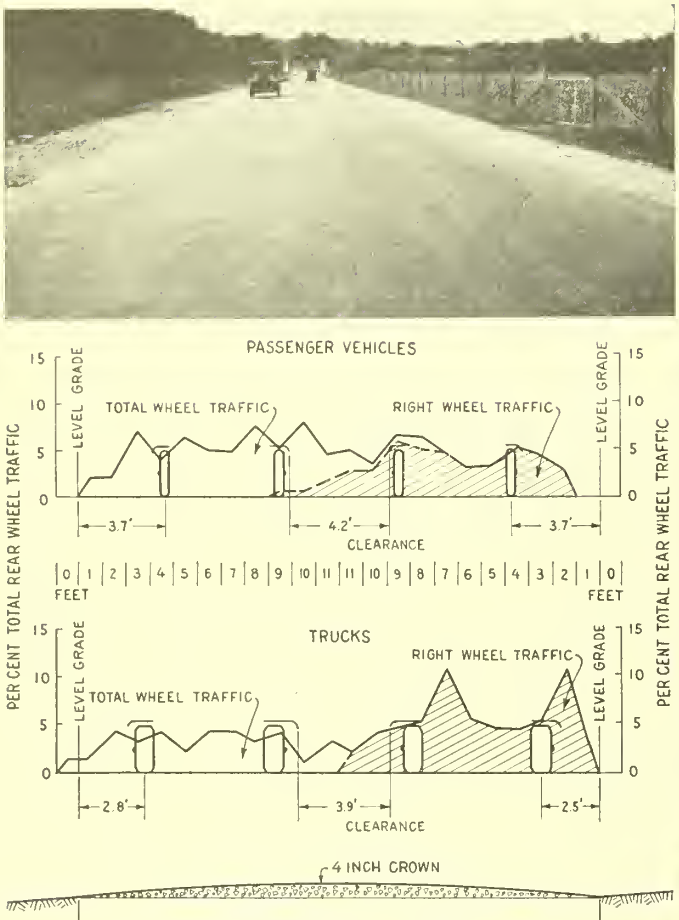

Fig. 6.—Test 23, made on a 22-foot bituminous macadam road, shows how the peaks of traffic concentration are reduced by widening the surface beyond 18 feet. The maximum percentage of passenger vehicle and truck concentration at one point are 8.1 and 11.1 respectively. The peaks are still further reduced by widening to 24 feet (see Fig. 7)

to the edge with a sense of safety than the drivers of passenger cars; and he prefers this position because there it is not necessary, as it would be closer to the center, to pull aside continually to permit the passing of faster cars. The drivers of passenger cars, on the other hand, possessed of a more mobile vehicle, apparently shuttle back and forth from the center to the side of the road, occupying the center when alone and pulling off to the side when passing. This is probably the reason why the peaks of passenger-vehicle wheel concentration are not so pronounced as the peaks in the motor-truck graphs.

THE POINT OF GREATEST TRAFFIC CONCENTRATION AFFECTED BY WIDTH OF ROAD

When width alone is the controlling factor it may apparently be anticipated that the greatest concentration of traffic on a narrow road will come upon the center of the roadway. This is due to the overlapping of the wheels of vehicles proceeding in opposite directions. Apparently this condition obtains for all widths up to and including 18 feet. Beyond that width the center peak of the distribution graph tends to become a valley and the peak of maximum wheel concentration is split into four lesser peaks, two of which appear on each side of the road.

Thus, in test 7 (Fig. 4) made on a 15-foot road the maximum percentage of the traffic concentrated on a single foot of width is 20.7 for truck traffic and 17.4 per cent for passenger-car traffic, each at the center of the road. The effect of widening the surface to 18 feet is shown by test 4 (Fig. 5) made on a level section of concrete road, which to all appearances is normal in every respect. The observations on this road show that both the passenger vehicle and truck traffic concentrate in four pronounced peaks, two on each side of the road, the maximum in the case of passenger vehicles representing 10.9 per cent of the total traffic, and in the case of trucks 12.7 per cent. What happens on a 24-foot road is illustrated by test 17 (Fig. 7), made on a granite-block pavement, where it is found that the peaks representing the left and right side concentrations are more widely separated and reduced to 7 per cent and 10.2 per cent, respectively, of the total traffic for passenger vehicles and trucks.

It thus appears that the width of the pavement is an essential consideration in interpreting the effect of a given total traffic on the pavement. Although the experiments of the Bureau of Public Roads indicate that surface wear of modern pavements by rubber-tired vehicles is practically negligible, the relative concentration of the traffic at various points on the surface, and the direction of the traffic may be found to be instrumental in the causing of other types of failures, such as fatigue and subgrade failures. If, as is generally suspected, the causation of corrugations in bituminous surfaces is related to the direction and volume of traffic using the road, it is possible that

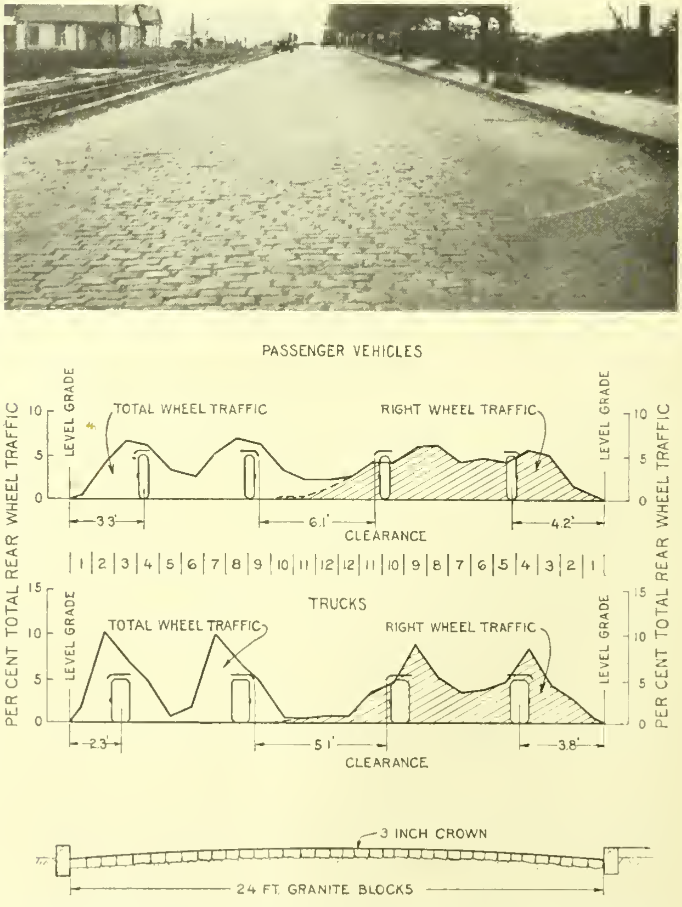

Fig 7.—Test 17, made on a 24-foot granite block pavement, shows the effect of widening the pavement on the concentration of traffic. The maximum peaks in this case are 7 and 10.2 per cent of the total traffic for passenger vehicles and trucks respectively. Shade trees at the right apparently cause the traffic to keep at a greater distance from the right-hand curb

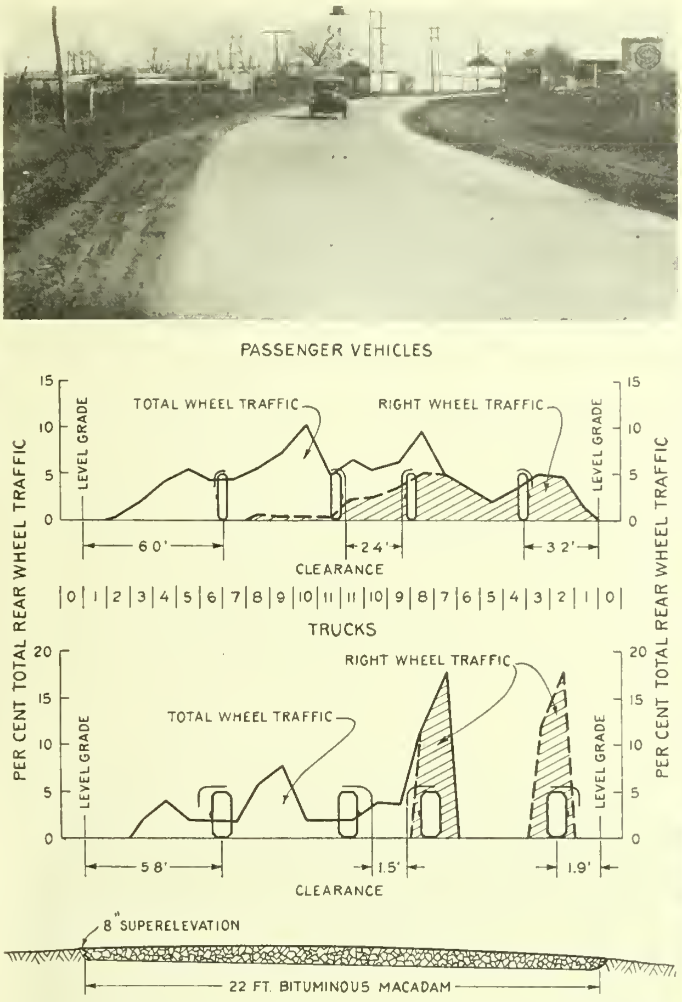

Fig. 8.—Test 18 shows the distribution of traffic on 22-foot bituminous macadam road at an 11° 30′ curve which has a crowned superelevated section. Vehicles traveling on the outside of the curve crowd toward the inside to take advantage of the favorable slope and reduce curvature, and in the absence of a center line there is nothing to counteract this tendency

certain commonly observed characteristics of such waves or corrugations may be explained by observations of lateral distribution. It is shown by the present tests that the center of the road almost always carries a traffic in which vehicles moving in opposite directions have their wheel tracks superimposed one upon the other; and it is possible that the disappearance of the waves at the center, which has been often observed, is due to the passage of approximately the same number of wheels in each direction, thus neutralizing the effect of direction of traffic.

THE EFFECT OF CURVATURE ON LATERAL DISTRIBUTION

Six of the points of observation were on horizontal curves. Two were on 24-foot bituminous macadam roads (tests 16 and 19), one with an 18 and the other with a 20 degree curve. Each had a white center line and both were superelevated, the 18-degree curve 9 inches with a crowned section and the 20-degree curve 6 inches with a plane section. The one 22-foot road (test 18) was also surfaced with bituminous macadam, and its 11-degree-30-minute curve was superelevated 8 inches with a crowned section which practically neutralized the effect of the superelevation in the outer quarter. This road had no center line and no other feature of any kind which might conceivably guide the traffic. Two 20-foot roads with curvature of 8 and 15 degrees (tests 13 and 14) introduced another element. Both were superelevated 5 inches with crowned sections and, as in the case of the 22-foot road, the crown neutralized the superelevation in the outer quarter. But both of these roads had 2½-foot concrete shoulders at each side of the 15-foot bituminous-macadam center section. Finally, test 22 was made on an 18-foot concrete road with curvature of 25 degrees, the sharpest of all the curves observed. The plane section had a superelevation of 5 inches and there was no center line; and the element which distinguished this curve from all the others, in addition to its heavy curvature, was a 5-foot fill at the outside of the curve, protected by a guardrail within 2 feet of the edge of the pavement.

In these six curves, then, we would expect to find intermingled the effects of each of the various elements; and unless the facts are kept clearly in mind, it will seem that there is no rational explanation of the striking differences which appear in the graphs. It is best to approach the detailed study by considering first the 22-foot, 11-degree-30-minute curve (Fig. 8). It is the simplest of the six. It has no center line; its shoulders on both sides are good; and there is a full view ahead from one end of the curve to the other. There are only two apparent reasons why the

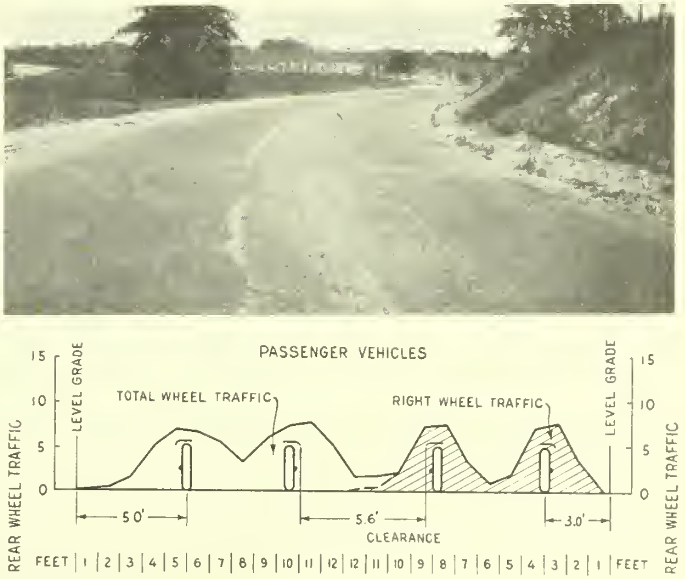

Fig. 9.—Test 16 was made on a 24-foot bituminous macadam road at an 18° curve. The curve is superelevated 9 inches with a crowned section and vehicles traveling on the outside move over to take advantage of the favorable slope, but are prevented from overlapping the center by the guiding line. The bank at the right causes the inside traffic to keep well out from the inner edge

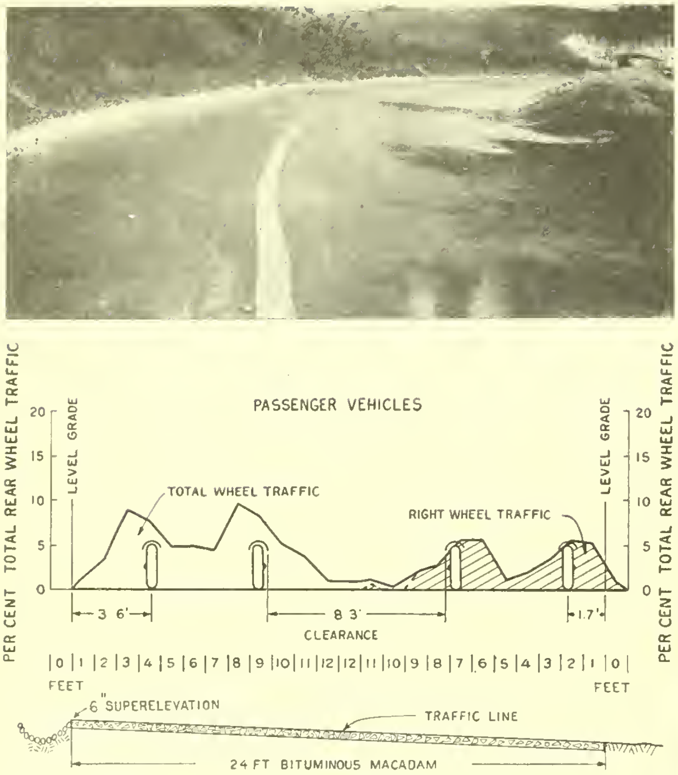

Fig. 10.—Test 19, made on a 24-foot bituminous macadam road at a 20° curve with a plane superelevated section and center line, shows a better separation of the traffic than test 16 (Fig. 9) made on a similar road with crowned superelevated section

traffic on this curve should shift from its preferred position on the tangent. These are the instinctive desire to cut across the curve and thereby lengthen the radius of the curved course; and the fact that the crowned section neutralizes the superelevation of the outer quarter of the road. If, therefore, the vehicles traveling on the outside of the curve are to get any benefit from the superelevation, their course must be shifted toward the inside of the curve so as to place them on the favorable tilt. This is apparently exactly what happens. The high point of the cross section if approximately 6 feet from the outer edge; and at this exact point we find the center of gravity of the outer wheel diagram, indicating that the average driving position of the vehicles—both trucks and passenger cars—is the position nearest to the outer edge at which the vehicle can be driven with full advantage of the superelevation. In taking this position the average position of the inner wheels of the outer traffic is brought practically to the center of the road in the case of passenger vehicles and slightly beyond it in the case of trucks. A very considerable percentage of the passenger-car traffic moves with the inner wheel more than 3 feet beyond the center, and the truck traffic crowds over nearly as far.

With this test in mind, let us now turn to the two 24-foot roads (Figs. 9 and 10). These two curves have practically identical conditions with one exception. The 18-degree curve has a crowned section and the 20-degree curve a plane section. Both are surfaced with bituminous macadam; both have center lines. The shoulder conditions differ slightly, in that one has a cobbled gutter at the inside of the curve and the other at the outside; but the gutters are shallow and the pavements are wide, and it does not seem likely, therefore, that this difference can have a material bearing on the behavior of the traffic. The major points of difference between the two curves are the type of superelevated section and the condition of the roadside at the inside of the curve. On the 18-degree curve the bank at the inside of the curve is high enough to obscure the view around the curve; on the 20-degree curve the forward view is not obstructed.

Both of the curves show clearly the separating influence of the center line. The advantage of the plane superelevated section is also indicated by the lesser distance from the outside edge of the pavement to the wheels of the vehicles on the 20-degree curve which is thus treated. On the 18-degree curve the average position of the rear wheels of the outer automobile traffic is found at 5 feet from the edge as compared with 3.6 feet on the sharper curve. On the 18-degree curve, also, the inside traffic operates at a greater distance from the inner edge; but this is undoubtedly due to the obscured vision.

However, the striking fact observable in the graphs of these two tests is the clear separation of the traffic on the left and right sides of the roads, which is doubtless attributable to the center lines. The overrun to the wrong side of the road which is the outstanding

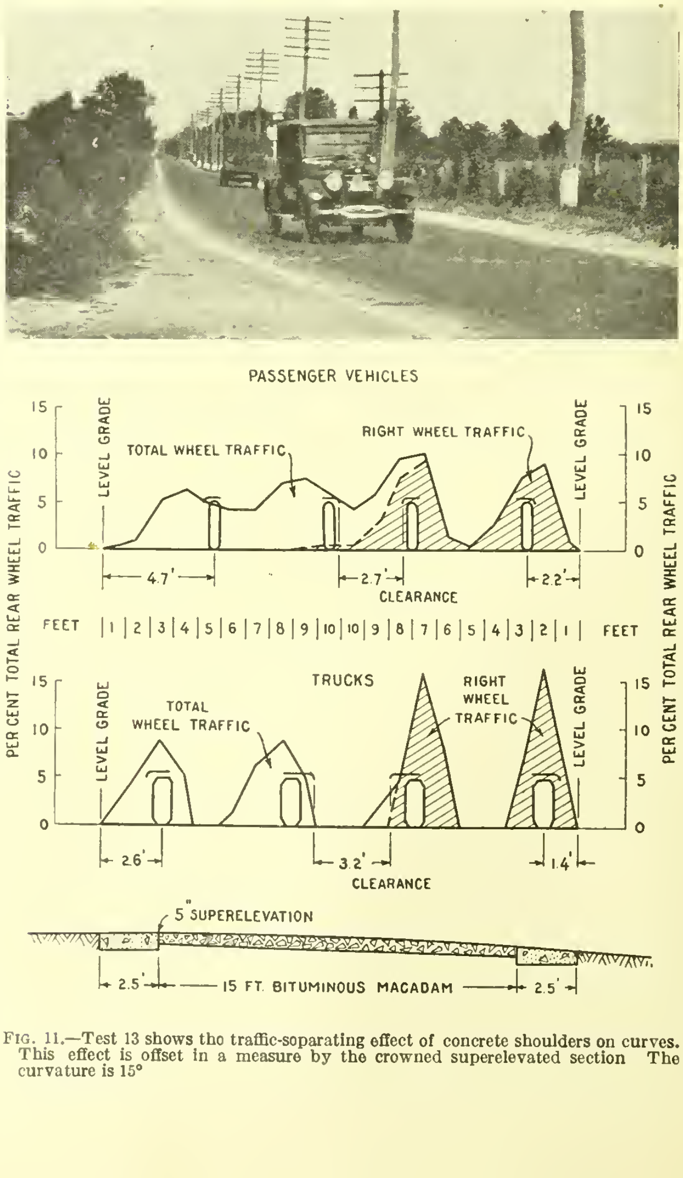

Fig. 11.—Test 13 shows the traffic-separating effect of concrete shoulders on curves. This effect is offset in a measure by the crowned superelevated section The curvature is 15°

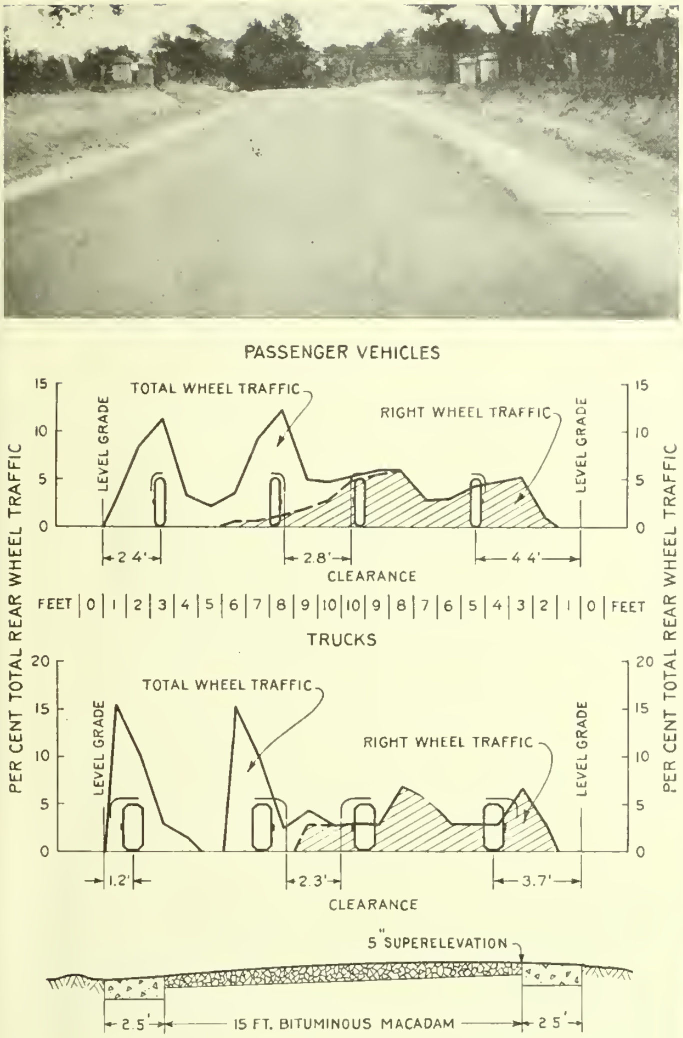

Fig. 12.—Test 14, like test 13 (Fig. 11), shows the contrary influences of concrete shoulders and a crowned superelevated section on the distribution of traffic on an 8° curve

characteristic of the graphs of the 22-foot road is almost completely absent in the graphs of these 24-foot curves; and the significance of the difference is heightened by the fact that the latter are almost twice as sharp as the former.

Obviously, therefore, in the comparison of the observations on these three roads there is strong argument for the plane superelevated section and a convincing demonstration of the effectiveness of the center line.

In the two 20-foot pavements (Figs. 11 and 12) we have an opportunity to observe a conflict between the contrary effects of a guiding line and a crowned superelevated section. The guiding line in these pavements is not a center line but an attractive white strip at the sides of their black surfaces. Obviously it is effective to a degree, for, on these pavements the outside traffic does not crowd toward the inside until it has both wheels on the favorable slope of the crown; but it is equally clear that the inner slope is attractive to a considerable percentage of the outside vehicles, especially the automobiles, some of which crowd over 4 or 5 feet beyond the center of the road.

Finally we come to the 18-foot curve (Fig. 13) which is different from all the others in these respects: It is paved with concrete throughout, it is much the sharpest,

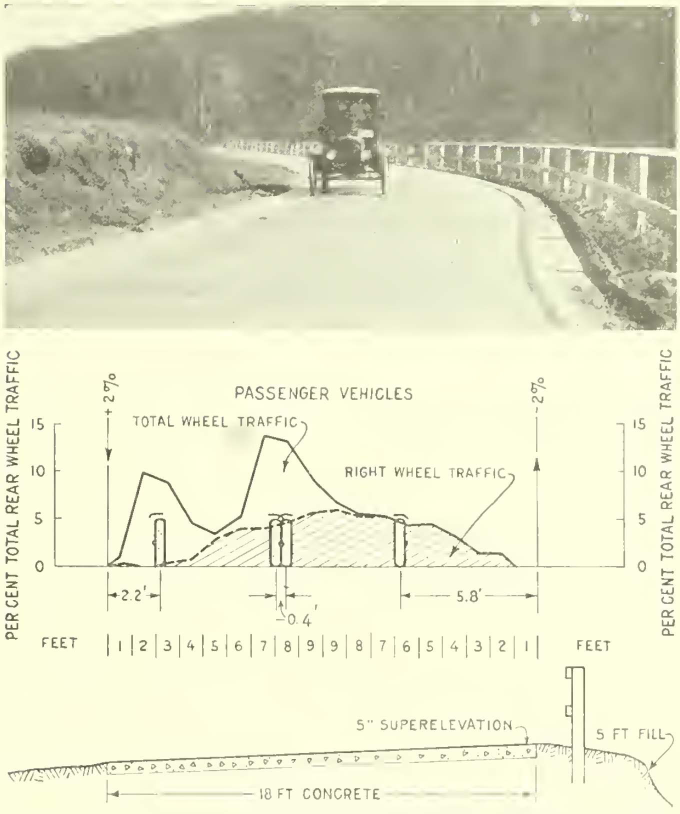

Fig. 13.—Test 22, made on an 18-foot concrete pavement at a 25° curve with plane superelevated section and no center line, shows the effect of the absence of the center line and the presence of a menacing fill at the outside of the curve on the distribution of traffic. The shift toward the inside of the curve is accentuated by the slight downhill grade at the outside

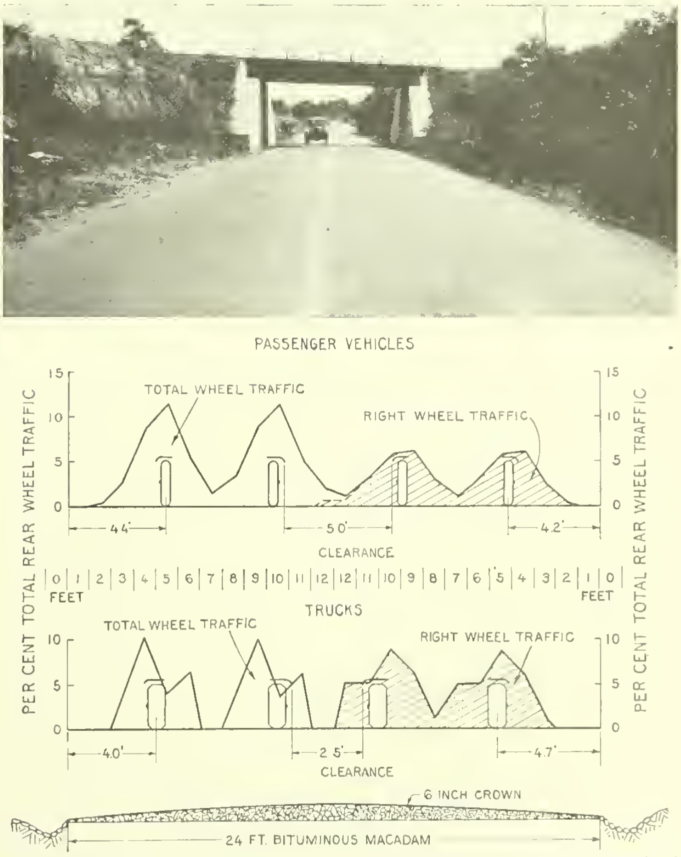

Fig. 14.—Test 12. The bridge abutments apparently reduce the effective width of this 24-foot bituminous macadam road to 20 feet, but the center line effectively separates the traffic

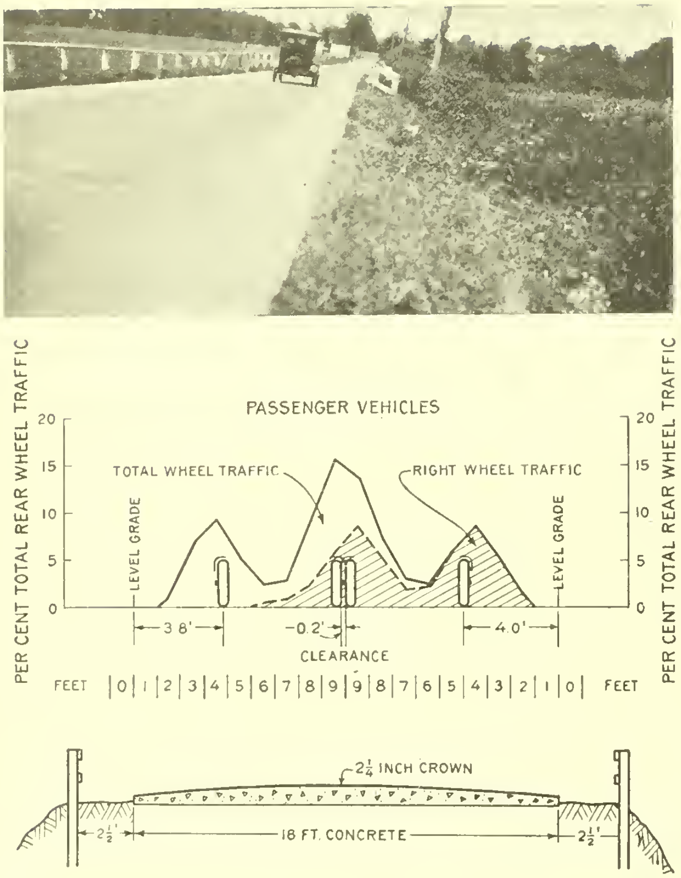

FIG. 15.—Test 3, made on an 18-foot concrete road, shows that the effective width is reduced to 16 feet by the low, narrow shoulder and menacing fill and guard rail

and it has at its outside a menacing fill which is guarded by a wooden fence that stands within 2 feet of the edge of the pavement. It is similar to one of the 24-foot pavements in having a plane superelevated section; it is similar to the 22-foot pavement in having no center line or other guiding element. The deviation of the automobile traffic from the proper legal course is more marked on this curve than on any of the others. Some of the outer traffic actually reaches the inner edge; the average position of the outer rear wheel is almost two-thirds of the distance from the outside to the center; and there is practically no clearance between the two lines of traffic. What is the explanation of these conditions? Undoubtedly the high degree of curvature is responsible in a large measure; for while the shift of course gains nothing in point of superelevation, there is an appreciable advantage to be gained on such a sharp curve in the way of increased radius of curvature. Moreover, the presence of the fill and guardrail at the outside of the curve, and the fact that the outside traffic is moving down grade, undoubtedly causes a movement away from the apparent danger. Add to these facts the further fact that there is no warning or separating line or other device to counteract the tendencies set up by the other factors, and we have a fairly satisfactory explanation of the behavior of the traffic on this road.

The observations are not numerous enough to reveal the general relation which probably exists between the degree of curvature, the tangent width of the pavement and the amount of widening on the curves which would

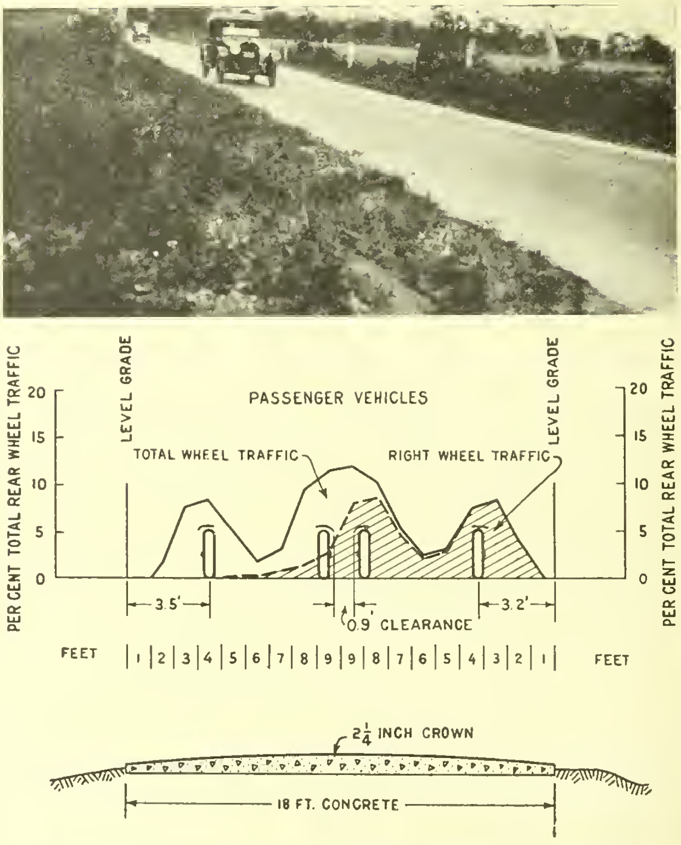

FIG. 16.—Test 1, also on an 18-foot concrete road, shows the loss of effective width caused by the low shoulders. Compare with roads of the same width but good shoulders shown in Figures 5, 17, and 18

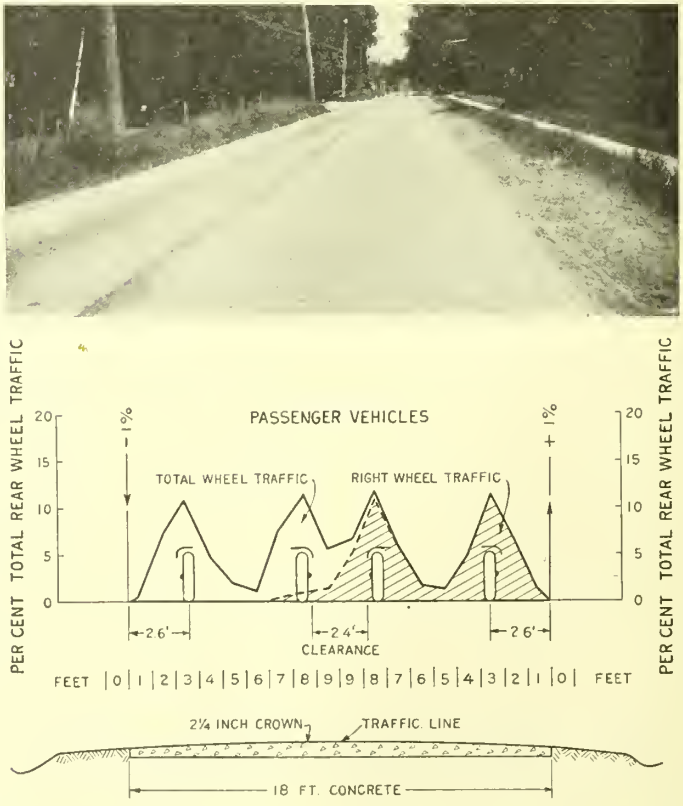

FIG. 17.—Test 11, made on an 18-foot concrete road with good shoulders. Compare distribution with that shown in Figures 15 and 16, which show the effect of bad shoulders

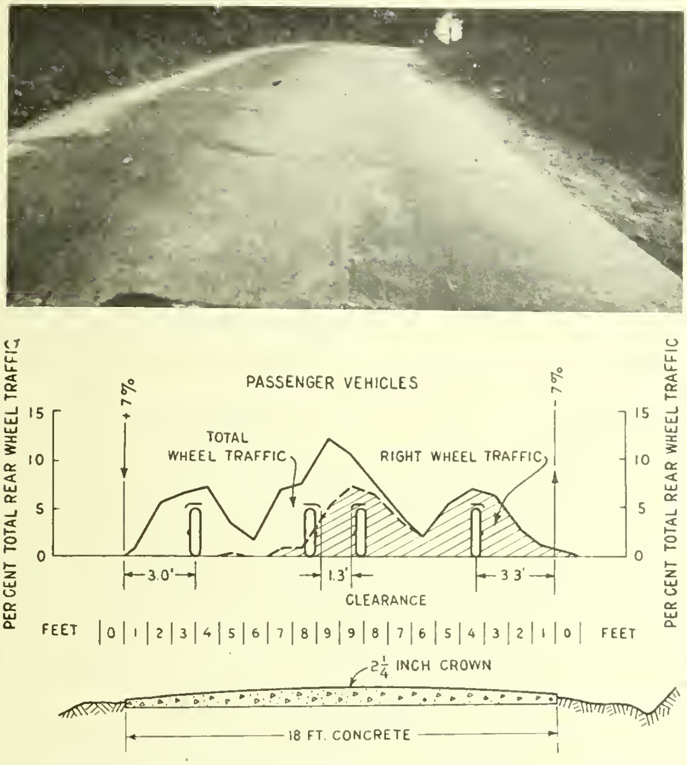

FIG. 18.—Test 20, also on an 18-foot concrete road, shows that the good shoulders permit full use of the surface, but the curve to the right just beyond the point of observation and the obstruction of the view of the right-hand traffic both cause a movement of the traffic toward the center, resulting in a center peak of traffic concentration. The presence of a center line would probably counteract this tendency

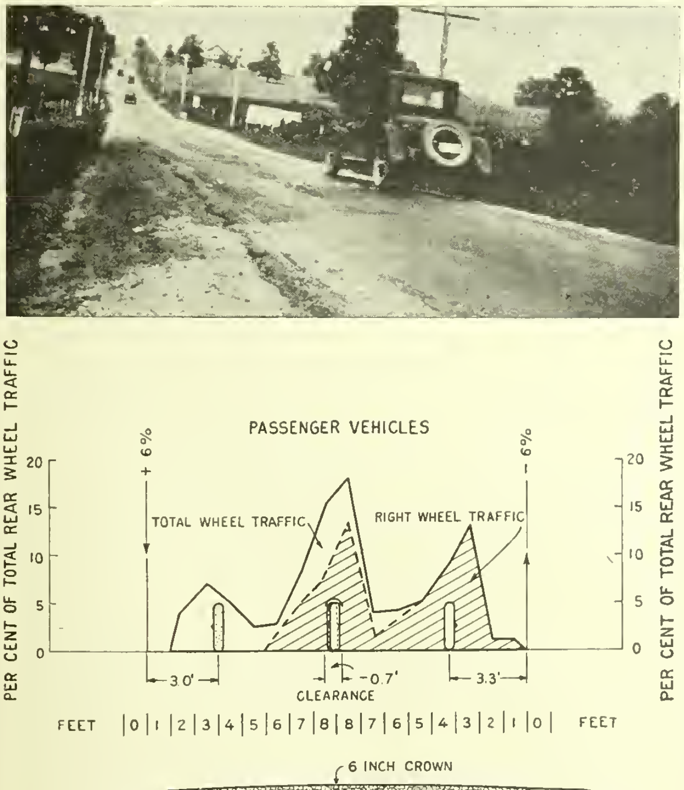

FIG. 19.—Test 8, a 16-foot bituminous macadam road, is effectively only 15 feet wide on account of the ragged edge at the left. The steep crown accentuates the tendency to crowd toward the center

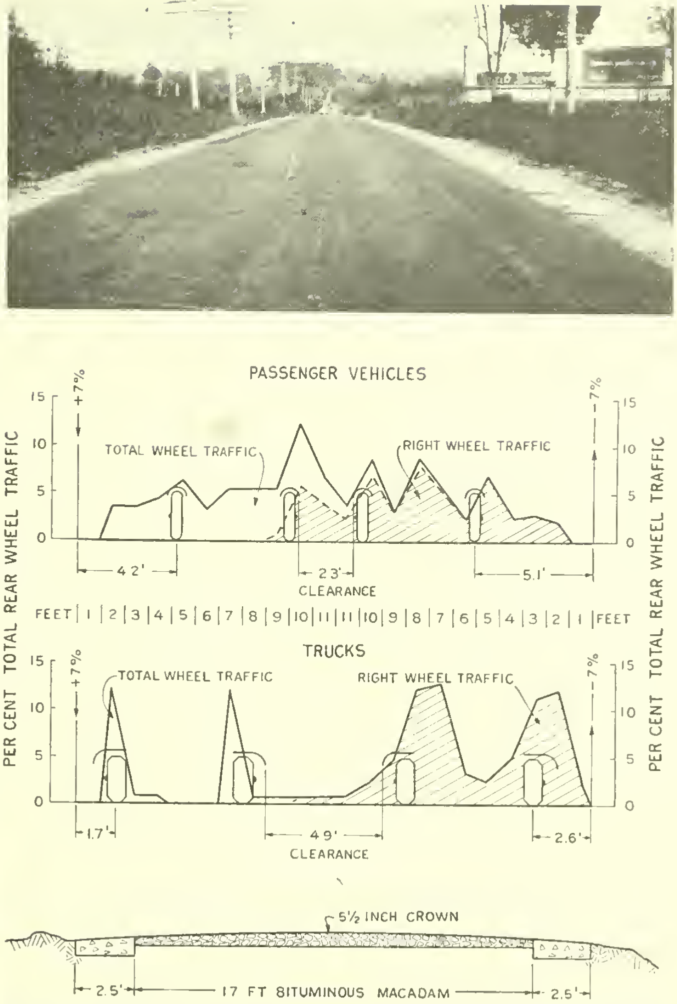

FIG. 20.—Test 21 shows how the effective width of a pavement may be reduced by defective surface some distance from the edge. To avoid the roughness of the surface approximately 6 feet from each edge the passenger vehicles keep their inner wheels nearer to the center and thus fail to utilize the full width of the road. The wider gauge of the trucks permits them to "straddle" the rough surface and still travel with their outer wheels at the edge

provide clearance equal to that provided on the tangent. The tests do indicate that the presence or absence of a center line, the character of the superelevated section and the roadside conditions are factors which should be considered as well as the degree of curvature and the tangent width in estimating the amount of widening required.

TRAFFIC MOVES TOWARD CENTER ON LIGHT DOWN GRADES

The only tendency clearly recognizable in connection with the distribution of the traffic on grades is a slight movement toward the center of the road on the downhill side of light grades. The tendency is not observed on heavy grades, but is apparent in practically every instance on grades of 1 or 2 per cent. The explanation which has been advanced to account for this phenomenon and which seems plausible, is that the vehicles are driven on light grades at practically the same speed as on the level, but that the drivers, sensing, the slightly increased danger, take the precaution of moving away from the edge. On heavy grades the drivers almost invariably reduce speed, and at the lower speed the instinctive fear of the edge is lessened.

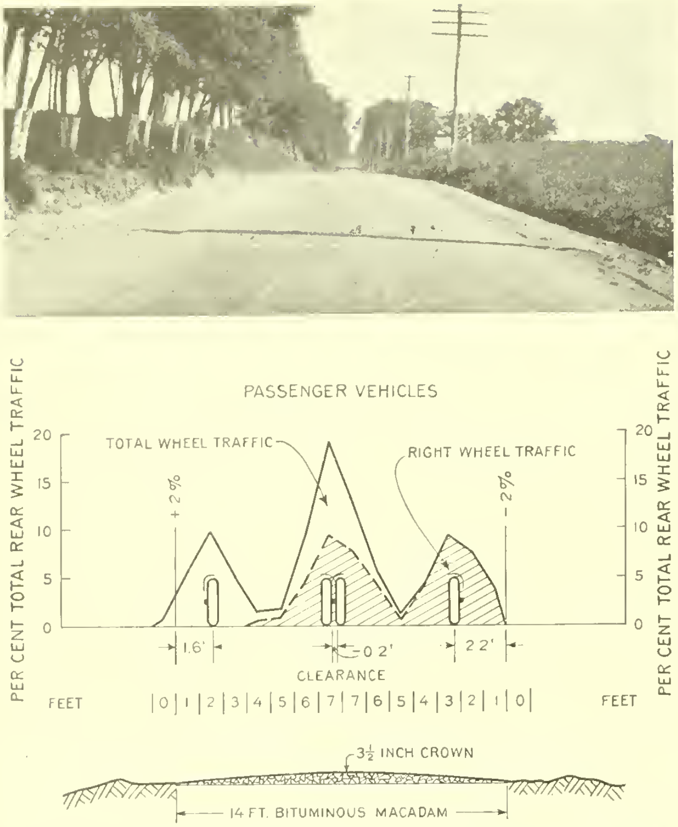

Fig. 21.—Test 9 shows the distribution of traffic on a 14-foot bituminous macadam road. Note that the traffic spreads out onto the shoulder at the left. This fact and the lower crown makes the negative clearance on this road less than that of the 15-foot road shown in Figure 4

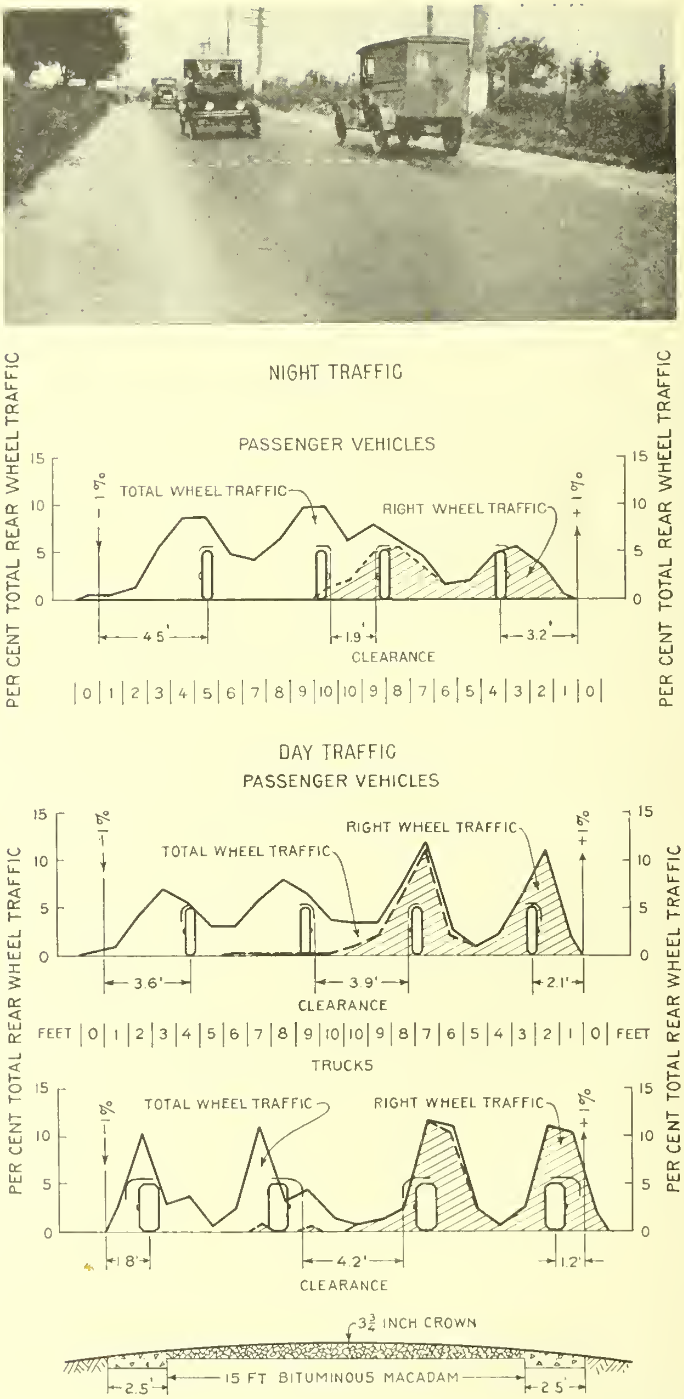

Fig. 22.—Test 5 shows the separating effect of concrete shoulders on tangents and difference in the distribution of traffic at night. Compare with the concrete road of the same width shown in Figure 23 which has a center line for the separation of the traffic

OTHER FACTORS AFFECTING LATERAL DISTRIBUTION

Among the most interesting results of the investigation is the evidence it supplies with regard to the effect of the condition of the roadside and the pavement itself on the distribution of the traffic and the effective width of the pavement. Reference has already been made to the effect of the bank at the inside of the curve observed in test 16. (Fig. 9.) By obstructing the forward view of drivers on the inside of the curve this bank has been shown to cause the inside traffic to move further in toward the center of the road than the traffic on the similar curve observed in test 19. (Fig. 10.) On the other hand, a 5-foot fill at the outside of the sharp curve observed in test 22 (Fig. 13) has been shown to accentuate the tendency of the traffic to move toward the inside of the curve.

It is clear from the graphs that similar roadside conditions have had a like effect upon the distribution of the traffic on tangents, an effect in some cases so pronounced that the effective width of the pavement is reduced from 2 to 4 feet. The effect is not merely to reduce the usage of the pavement near the edges, but, apparently, to positively discourage all use of the pavement near the edges.

In test 12 (Fig. 14), for example, the heavy concrete abutments of an overhead railroad bridge erected at the edge of a 24-foot pavement, with only a narrow cobble gutter between, are shown to so reduce the usage of the outer 2 feet of the pavement at each side as practically to convert it at this point into a 20-foot pavement.

Test 3 (Fig. 15), made on an 18-foot concrete road, shows that the effective width of the pavement is reduced to 16 feet by the presence of a low, narrow shoulder and menacing fill and a guardrail only 2½ feet from the edge of the pavement. Almost as much effect is attributable to the low shoulder observed in test 1 (Fig. 16), also on an 18-foot concrete road. Comparing the observations on these two roads with those on the concrete roads of the same width treated in tests 4, 11, and 20 (Figs. 5, 17, and 18) brings out clearly the fact that it is the roadside condition which has caused the reduction in effective width in these cases. Figures 17 and 18 representing like conditions of character and width of surface and shoulder illus-

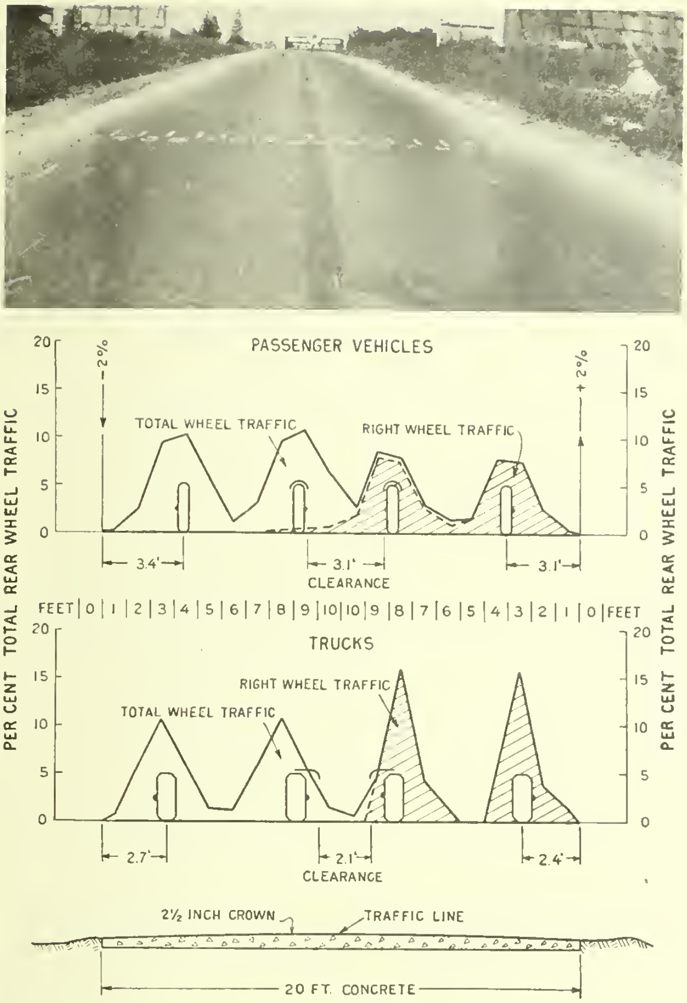

FIG. 23.—Test 6, made on a 20-foot concrete road with center line, shows the separating effect of the black line. Compare with Figure 22 which illustrates the effect of concrete shoulders. Also note how the center of the oil marks coincides with the center of the average position of the vehicles

trate the effect of a center line on the position of the traffic on tangents.

In test 8 (Fig. 19) the road observed has a bituminous macadam surface 16 feet wide which is reduced to an effective width of 15 feet by the ragged edge of the surface on the left-hand side; and a similar result follows from a different surface defect in the pavement observed in test 21 (Fig. 20). Here the defective surface is not at the edge but approximately 6 feet from it on each side. Along a line at about that distance from the edge the surface is so rough as to discourage its use by the traffic. The edges of the pavement are surfaced with concrete and are smoother than any part of the central bituminous macadam section, but because of the specially rough condition 6 feet from the edge it appears that the drivers of passenger vehicles will not travel with their inner wheels at this point, with the result that the outer foot of the concrete shoulder on each side is not used. The wider wheel gauge of the trucks permits them to drive with their outer wheels on the concrete near the edge without placing the inner wheels on the rough surface, and the graph shows that they do utilize the concrete shoulder.

EXCESSIVE CROWN CONCENTRATES TRAFFIC AT CENTER

The effect of excessive crown is indicated by comparison of tests 7 and 9 (Figs. 4 and 21). Test 7 was made on a 15-foot bituminous macadam surface with a crown of 1 inch per foot. Test 9 was made on a similar surface, 14 feet wide with a crown of one-half inch per foot. It is apparent that the greater crown of the wider road has so concentrated the traffic at the center as to reduce the clearance to a negative distance of 1.2 feet as compared to a negative 0.2 foot on the narrower road.

The effect of darkness on the distribution of traffic is illustrated by test 5 (Fig. 22). Observations were made on this road during the night as well as in daylight. It was anticipated that the white concrete shoulders, being more visible at night than the black center section would have the effect of holding the traffic to the edges of the pavement. Comparison of the graphs of day and night observations, however, shows that this result is not obtained to any marked degree. It will be noted that the density of the traffic at the center, which appears as a valley in the graph of the daylight observations, forms a peak in the graph of the night observations. No observations were made on other pavements at night and it is therefore impossible to say definitely that the concrete shoulders have not been effective in some degree. It is possible that on an all-black surface the center concentration might have been found to be even greater.

The comparative effects of concrete shoulders and a center line in separating traffic on tangents are illustrated by comparing with test 5, the results of test 6 (Fig. 23) made on an all-concrete pavement, also 20 feet wide. In relation to the motor-truck traffic especially the concrete shoulders apparently exert a greater influence than the center line.

H. F. JANDA APPOINTED ASSISTANT DIRECTOR OF HIGHWAY RESEARCH BOARD

Announcement is made by Charles M. Upham, director of the highway research board of the National Research Council, of the appointment of H. F. Janda as assistant director. Mr. Janda is especially fitted for this position. He is a graduate of the University of Wisconsin and for two years was assistant city engineer at Portage, Wis. Mr. Janda has not only had a variety of practical experience in engineering but has also acted as instructor and assistant professor of civil engineering at the University of Cincinnati for five years. As associate professor of highway engineering at the University of North Carolina for three years he was in charge of experimental research in cooperation with the State highway commission. Mr. Janda has carried on many important research projects, including studies of capillary moisture in highway subgrades, earth-pressure tests on culvert pipe, and other problems affecting highway construction.

Mr. Janda is located in the offices of the highway research board in the building of the National Academy of Sciences and National Research Council at B and Twenty-first Streets, Washington, D. C.

THE EFFECT OF MOISTURE ON CONCRETE[1]

By W. K. Hatt, Professor of Civil Engineering, Purdue University

THIS paper reports investigations conducted by the Engineering Experiment Station of Purdue University in cooperation with the United States Bureau of Public Roads,[2] the primary purpose of which was to measure the maximum warping and surface deformation of a concrete road slab resulting from nonuniform distribution of moisture, as a basis for estimating possible initial stresses. The tests also yield data of expansion and contraction of concrete, useful in the design of concrete structures.

The investigations were divided into two sections, namely: (1) Observations on a concrete road slab, 7 inches thick, 18 feet wide, and 25 feet long, resting on a porous subgrade, and covered with a tent and housed in a shed to control the distribution of moisture and the effects of temperature; (2) tests of strength and length changes of small beams, some partly and some uniformly saturated.

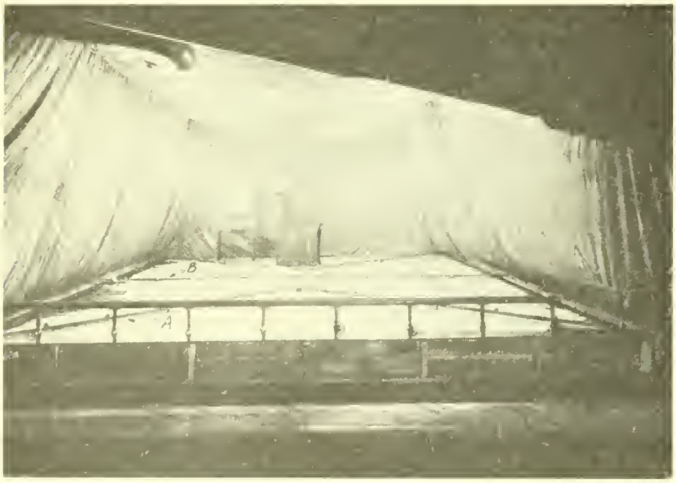

FIG. 1.—General view of concrete test slab showing tent covering and means of saturating subgrade

The following phenomena were examined:[3]

1. The maximum distortion (including deflection and fiber deformation) of the concrete slab, caused by nonuniform distribution of moisture.
2. The effect of moisture changes on the modulus of rupture and compressive strength of concrete determined by tests of the beams, the dimensions of which were 4 inches by 7 inches by 4 feet, and cylinders 6 inches in diameter by 12 inches in length.
3. The volume changes in the concrete beam specimens due to exposure, including warping due to partial saturation.

CONCLUSIONS DRAWN FROM THE TESTS

From the results of the various studies the writer has reached the following conclusions:

1. The strength of concrete, like that of wood varies with its moisture content. While the law relating to moisture content and to strength was not determined throughout various ranges of moisture content, it appears that saturated concrete will have from 80 to 85 per cent of the strength of dry concrete.
2. The thermal coefficient of expansion of concrete varies with the temperature and moisture condition of the concrete.
3. Concrete expands when immersed in water and contracts on drying. The degree of change of length varies markedly with the characteristics of the brand of cement, with the richness of the mix, with the size of specimen, and with the conditions of exposure. Measurements begun after the initial hardening of the cement show that concrete may expand 0.01 per cent due to the absorption of moisture, and may contract 0.06 per cent when exposed to dry air. These changes continue. To these values should be added the changes in length during the initial setting of the cement.
4. Concrete road slabs warp upward at the corners and at the edges when the surface becomes dry, and also when the bottom absorbs moisture from the subgrade. Drying the top surface of the slab under observation deflected the corners upward 0.12 inch. When the bottom of the slab was saturated from a water-filled subgrade the upward corner deflection reached 0.20 inch.
5. This curling upward of the corners of the road slab presents a cantilever beam to the load of a passing truck. The results of the tests indicate the presence of an initial stress in the surface of the road slab arising from the warping.
6. A combination of shrinkage from drying and from a fall in temperature will produce maximum shrinkage strains. A combination of a drop in temperature and a rainfall may retain the concrete at its previous length. The friction of a road slab on its subgrade is an element favoring the production of cracks.
7. To be durable, concrete structures should be designed for the least favorable conditions. Stresses arising from the effect of temperature and moisture as well as those due to dead and live loads should be considered.
8. Without careful curing, concrete masses will undergo excessive surface shrinkage against their rigid cores, thereby favoring surface cracks. Therefore steel reinforcing should be placed near enough to the surface to withstand these shrinkage strains, and at the same time far enough below the surface to be protected against corrosion.
9. Foreign experiments show the benefit of early and careful curing of concrete in reducing final shrinkage. The greater part of the shrinkage of mortar specimens has been found to take place in the first 10 hours. These foreign experiments also show the greatly increased shrinkage of mortar mixed with an excess of water.
10. Since the results of measurements of expansion vary with several factors, such as the size of specimen, the time of initial measurement, and the exposure, the method of such tests should be standardized.

The writer believes that the following are the requirements for concrete structures exposed to the weather:

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[1] The substance of a paper presented at the meeting of the highway division of the American Society of Civil Engineers, New York, N. Y., Jan. 22, 1925.

[2] The tests were made under the supervision of the writer and Prof. R. B. Crepps. A. C. Benkelman was in direct charge, assisted by E. Gustavsen, both of the United States Bureau of Public Roads. R. E. Mills, of the Engineering Experiment Station of Purdue University, also assisted from time to time. Special acknowledgment is made of the painstaking care in observation and the devotion to the tedious program of measurements contributed by Mr. Benkelman.

[3] The approximate number of observations involved in the entire study was: Deformation, 40,000; deflection, 3,000; strength, 600; weight, 1,000; and temperature, 1,500.

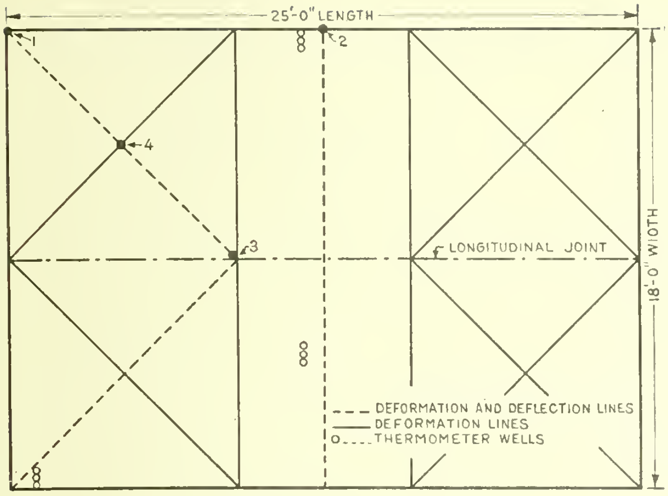


FIG. 2.—Plan of slab and observation layout

(*a*) A Portland cement of normal volume constancy and a clean, well graded aggregate.

(*b*) An impervious concrete of proportions not less than 1:2:4.

(*c*) A time of mixing of not less than 1 minute and preferably 1½ minutes for the production of uniform concrete.

(*d*) A concrete mixed with a minimum amount of water appropriate to the work at hand.

(*e*) Careful slicing of this plastic concrete against the face of the forms so that the reinforcing steel may not be exposed nor the surface require patching.

(*f*) Early curing begun just after the initial set and continued for a time depending on local conditions.

(*g*) Design of reinforcing steel and contraction or expansion joints to prevent cracking and the entrance of water.

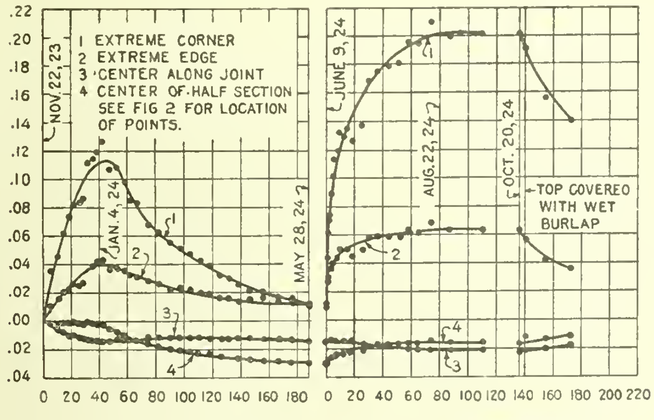


FIG. 3.—Deflections of road slab

DESCRIPTION OF THE TEST SLAB

The one-course concrete slab, made of 1:1½:3 concrete, was 7 inches thick, 18 feet wide and 25 feet long, with a longitudinal doweled center joint. A subgrade, consisting of a 4-inch layer of coarse gravel, was held in an impervious wooden subbase, the sides of which extended above the surface of the slab. The slab was housed in a shed and further protected by a tent. By means of perforated pipes in the gravel subgrade, through which warm air could be admitted, the subgrade could be dried; and water could be admitted to wet it again when desired. Warm, dry air was maintained above the slab in the tent enclosure.

Previous to the construction of the slab, concrete bench marks, for the support of the beams from which deflections were measured, were set 4 feet below the level of the ground. Rods extended upward from the bench marks through pipes set in the subgrade and slab. The slab and observation layout and the tent coverings are shown in Figures 1 and 2. Deformation measurements were made with 20-inch Berry strain guages; and deflections were measured with Ames dials reading to 0.001 inch.

THE EFFECT OF NONUNIFORM DRYING

The slab was cast on September 12, 1923, and was cured under water for 30 days. Following the curing period hot, dry air was forced under the slab through the perforated pipes in the gravel fill by a pressure fan; and, simultaneously, a drying atmosphere was maintained above the slab in the tent inclosure. Observations of the subsequent deformations and deflections, taken at weekly intervals, are shown in Figures 3, 4, 5, and 6.

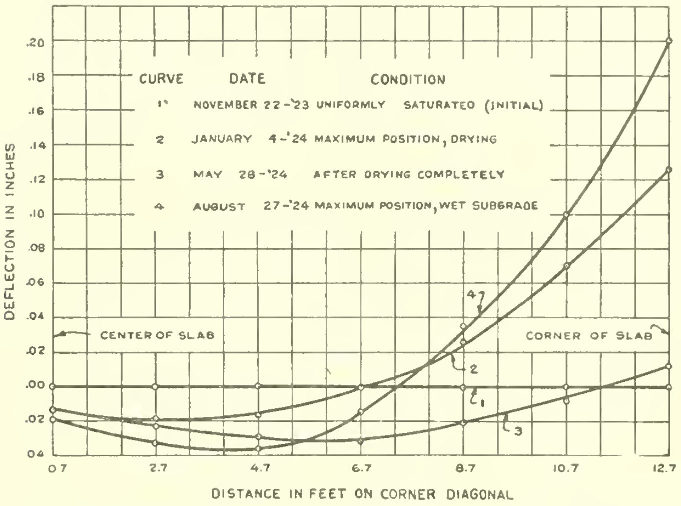


FIG. 4.—Typical profiles of the road slab showing effect of nonuniform moisture content

The more pronounced shrinkage occurring in the upper fibers was manifested by the fact that the edges and corners gradually curled up. A maximum uplift of 0.04 inch for the edges (at point 2, Fig. 3), and 0.12 inch for the corners (at point 1, Fig. 3), was attained at the end of 40 days. The subsequent return to a nonwarped or uniformly dry condition required five months. A slight permanent distortion as shown at point 3, Figure 4, representing the diagonal profile previous to the saturation of the subgrade is attributed to the more pronounced contraction of the richer troweled surface than the body of the slab. Observations were made of the longitudinal, transverse, and diagonal deformations of the upper surface of the slab. An average of two corner diagonals is shown in Figures 5 and 6.

All measurements of surface deformation of the slab were corrected for the changes in temperature which took place throughout the concrete. These temperature changes took place uniformly, that is, the readings of warping caused by the moisture gradient in the slab, were always taken when the temperature was uniform

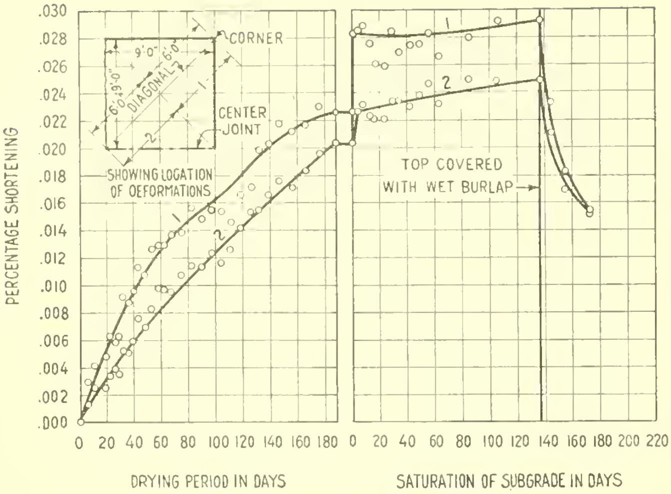


FIG. 5.—Deformation of upper surface of slab

throughout the depth of the slab; but there might be a rise or fall in temperature between the readings of two successive weeks. In this case the deformation readings were corrected for the temperature on the basis of a coefficient of expansion appropriate to the temperature of the concrete. (See Fig. 7.) The deformations shown in Figure 5 are therefore the net values after the corrections have been applied. Figure 6 shows the actual and corrected deformations, together with the changes in the temperature of the slab (uniform throughout the depth of the concrete). As the temperature changes were uniform throughout the depth of the slab there appears to be no reason for any correction of the vertical deflections caused by the moisture gradient.

The total diagonal contraction due to the moisture gradient between the condition of saturation (November 22, 1923) and that of completely dried concrete (May 28, 1924) was 0.021 per cent (Fig. 6). This value, like all others, has been corrected for uniform temperature changes. As will be shown later, beams of 1:1½:3 concrete contracted 0.022 per cent in 160 days. Hence, it appears that friction of the subgrade did not restrain the movement.

The fact that some of the points fall off the curve in Figure 3 is attributed to a slight effect of temperature in the warping of the slab, that is, they indicate that the contemplated procedure which called for the taking of observations only at times when the temperatures were uniform from top to bottom of slab, was not completely complied with.

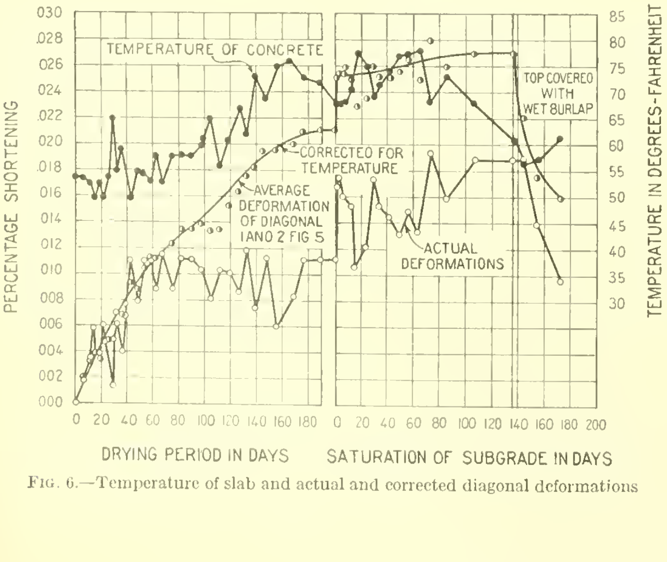


FIG. 6.—Temperature of slab and actual and corrected diagonal deformations

THE EFFECT OF NONUNIFORM SATURATION AND TEMPERATURE VARIATIONS

Following the drying period, water was introduced into the gravel subbase and maintained level with the lower surface of the slab. Differential expansion from bottom to top was again manifested in an immediate and continued gradual curling up of the edges and corners. A maximum distortion of 0.066 inch for the edges and 0.202 inch for the corners was attained at the end of 3 months. Typical results of the observations from June 2, 1924, the date of starting the saturation tests, until October, 1924, are shown in Figures 3, 4, 5, and 6.

While the subgrade was being resaturated the surface contracted diagonally 0.007 per cent in 110 days as shown by Figure 5. Of this contraction 46 per cent took place in 2 hours and nearly 100 per cent in 2 days. The shortening was caused by the expansive force of the saturated lower part of the slab, and occurred without a change in moisture content or temperature of the top layers. Assuming an elastic condition and a modulus of elasticity of 5,000,000 pounds per square inch, a compressive stress of 350 pounds per square inch in the top is indicated.

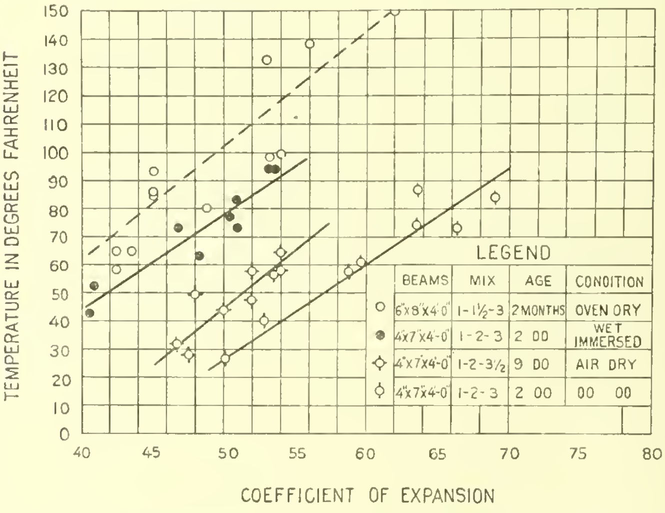


FIG. 7.—Thermal coefficient of expansion of concretes at different temperatures

When the slab had reached a static condition with saturated subgrade (on Oct. 20, 1924), an attempt was made to expand the surface by applying saturated burlap to simulate the effect of prolonged rain. In 30 days the corner had dropped 0.060 inch to a deflection of 0.140-inch (see Fig. 3), and the diagonal deformation had been reduced 0.014 per cent (Fig. 5).

The effect of daily variations of temperature in the air above the slab (not the outdoor temperature) is shown in Figure 8. A rise in temperature of 41° F. from 8 a. m. to 3 p. m. warped the corner downward 0.023 inch, or 21 per cent of the total warping (0.11 inch) at that period. While the air temperature changed from 34° F. to 75° F., the temperature of the top of the slab changed from 44° F. to 49° F., and that of the bottom of the slab from 47° F. to 49° F., a temperature differential of 3° F., which was the cause

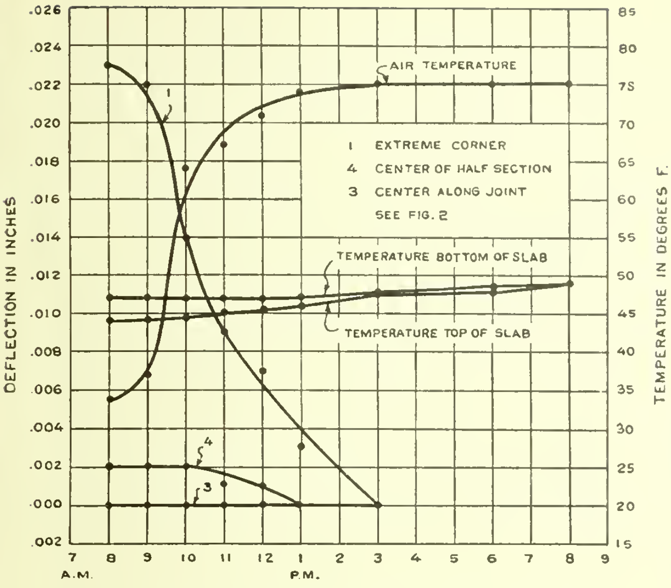


FIG. 8.—Deflection of road slab caused by temperature changes

of the warping. This warping may be compared to the deflection of 0.202 inch caused by the nonuniform saturation of the slab by the wet subgrade.

LOAD TESTS ON THE SLAB

Known loads were applied to one corner of the slab by jacking at three stages of the test: (1) On June 14, 1924, during the drying stage; (2) on September 23, 1924, during the saturation of the subgrade; and (3) on November 14, 1924, when the top had been saturated 25 days with wet burlap. The profiles of the diagonal on these dates for a load of 3,000 pounds are shown in Figure 9. The deflection data are given in Table 1. The deformation along the diagonal under the three conditions varied from 0.000031 inch per inch to 0.000037 inch per inch, the greatest deformation corresponding to the load applied during the drying period and the least to the load applied when the surface had been saturated with wet burlap.

As the warping of the slab increased (see (2), Fig. 9) the point of support must have moved in, increasing the deflection under load. Curiously enough, the deflection of the slab under load when the top was wet was less than in either of the other two cases.

TESTS OF CONCRETE MATERIALS

Six brands of cement were tested in connection with the various investigations of beams, compressive cylinders, and the road slab. These brands are designated in the reports of the tests which follow, as A, B, C, D, E, and F.

In the road slab cements A and D were mixed in equal parts and combined with sand and gravel in the proportions of 1:1½:3 for one half-section of the slab. For the other half, brand F was mixed with the same aggregates in the same proportions. In some of the beams cement F was used with the same aggregates in the proportions of 1:2:3½. The results of the usual standard tests of the cements used in the beams and slab are given in Table 2.

TABLE 1.—*Deflection of slab due to corner load of 3,000 pounds*

	Condition		
	During drying	Bottom wet	Top wet
Corner level, in inches:			
Before load	0.0131	0.0183	0.0124
With load	.0077	.0120	.0091
Deflection due to load	.0054	.0063	.0033
Set due to load, after 5 minutes	.008	.009	.008
Percentage of elongation under stress (in distance of 48 inches adjacent to the corner):			
Diagonal	.0037	.0035	.0031
Edges	.0025	.0023	.0017

The aggregates were washed sand and gravel from the Western Indiana Sand & Gravel Company, at La Fayette, Ind. The results of the sieve analysis and miscellaneous tests of the aggregates are given in Table 3.

The coarse aggregate was separated into three sizes: Between the 2-inch and 1-inch screens; between the 1-inch and ½-inch screens; and between the ½-inch and ¼-inch screens. It was then recombined in such proportions as to give maximum density according to Fuller's maximum density curve. The consistency of the mix was checked by the slump-cone method. All materials, including water, were weighed for each batch. The properties of the concrete are listed in Table 4.

Expansion and contraction of cements.—Changes in the length of specimens made of the six brands of cement neat, and a mixture of brands A and D, and concrete specimens made with brand F and the mixture of A and D, were made with Berry extensometers. The specimens of neat cement were 2 by 2 by 24 inches, those of concrete were 4 by 7 inches by 4 feet. The measurements were made for the purpose of ascertaining the expansion and contraction of the specimens on immersion in water and subsequent exposure to air. Measurements began two days after moulding, with previous moist-chamber storage. The results are shown in Table 5 and Figures 10, 11, and 12.

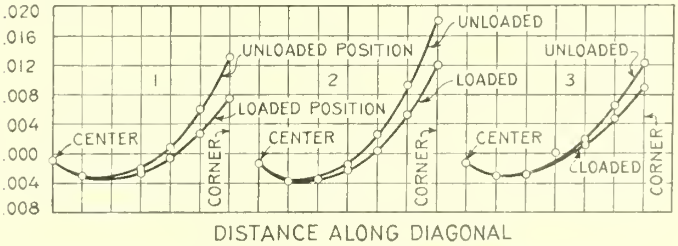


FIG. 9.—Deflections of corner diagonal of slab under 3,000-pound corner load

TABLE 2.—*Tests of cement*

Brand	Percentage of water		Percentage retained on 200-mesh sieve	Time of setting				Soundness	Tensile strength	
	Normal consistency	Mortar		Initial		Final			7 days	28 days
				Hrs.	*Min.*	*Hrs.*	*Min.*		*Lbs.*	*Lbs.*
A and D mixture, equal quantities	24	10.5	[1] 17.17	2	10	5	15	O. K.	297	373
D	23	10.3	[1] 20.3	2	35	5	20	O. K.	242	326
A	24	10.5	[1] 22.6	4	00	5	55	O. K.	255	327
F	25	10.7	19.1	3	35	5	15	O. K.	220	333

[1] Different samples.

TABLE 3.—*Tests of aggregates*

Unit weight of coarse aggregate =102.5 pounds per cubic foot.
Unit weight of fine aggregate =106.8 pounds per cubic foot.
Fineness modulus of coarse aggregate =7.32.
Fineness modulus of fine aggregate =3.17.
Fineness modulus of combined aggregate =5.90 for a 1:1.5:3 mix.
=5.77 for a 1:2:3.5 mix.

SIEVE ANALYSIS OF GRAVEL

(Tyler standard sieves)

	Total percentage retained on each sieve								
Sieve sizes	1½ in.	¾ in.	⅜ in.	No. 4	No. 8	No. 14	No. 28	No. 48	No. 100
Percentage	0	46	87	98	99	99	99	99	100

SIEVE ANALYSIS OF SAND

(Tyler standard sieves)

	Total percentage retained on each sieve						
Sieve sizes	⅜ in.	No. 4	No. 8	No. 14	No. 28	No. 48	No. 100
Percentage	0	0	3	38	77	97	99

TABLE 4.—*Tests of concrete*

Slump (4 to 8 by 12-inch truncated cone) =1 to 1½ inch.
Water-cement ratio =0.809 for a 1:1½:3 mix.
Water-cement ratio =0.935 for a 1:2:3½ mix.
Specific gravity =2.68.
Density =0.896.
Percentage of voids =10.4.
Percentage of absorption by weight =4.29.
Average unit weight, both concretes:
Oven dried to constant weight =149.89 pounds per cubic foot.
Immersed in water to constant weight=156.32 pounds per cubic foot.
Air dried in laboratory for 135 days=151.89 pounds per cubic foot.

Expansion due to temperature.—The coefficient of temperature expansion of four concretes was measured, with the results shown in Figure 7. The temperature of the concrete was obtained from blank beams in which there were thermometer wells consisting of steel pipes filled with a light paraffin oil. The thermometers were graduated to one-fifth of a degree. Reading from the left, the chart covers: (1) Three beams 6 inches by 8 inches by 4 feet, 1:1½:3 mix, age 2 months, cured under wet burlap during 30 days, and dried in oven at 150° F. for 21 days; the beams were taken from the oven and readings taken at 20° F. intervals as the temperature dropped, four cycles were read on as many days. (2) Two beams, 7 inches by 4 inches by 4 feet, 1 : 2 : 3 mix, age 2 months, kept immersed in water the temperature of which was varied and held constant 2 hours before observation. (3) Three beams, 4 inches by 7 inches by 4 feet, 1 : 2 : 3½ mix, age 9 months, cured under wet burlap for 14 days oven-dried for 1 month and air-dried for 7½ months; beams were moved from warm laboratory air to cold air of shed. (4) Two beams, 4 inches by 7 inches by 4 feet, 1 : 2 : 3 mix, age 2 months, cured under wet burlap for 14 days, in air 6 weeks. The various curves indicate that the variation of the coefficient of thermal expansion varies with the temperature range under observation.

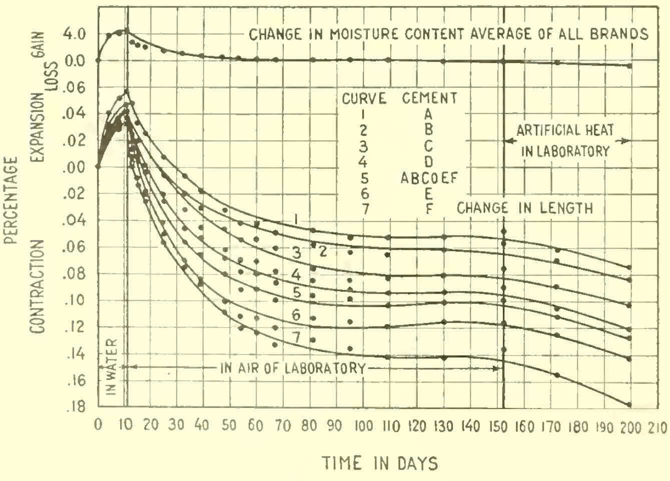


FIG. 10.—Contraction of neat cements, air-dried

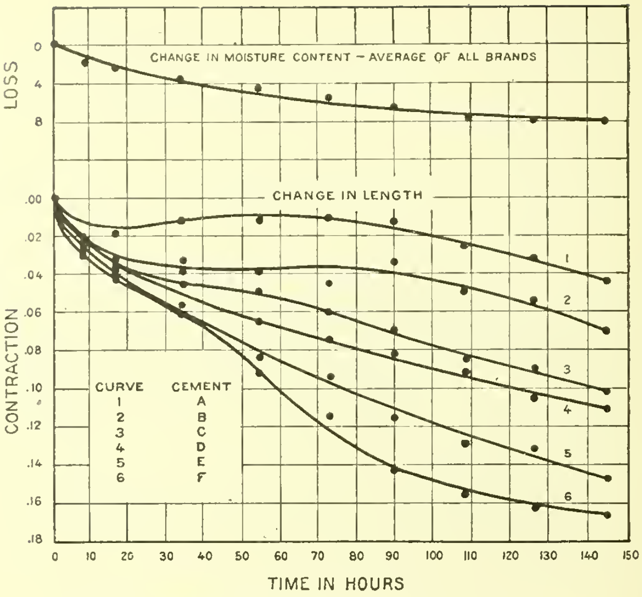


FIG. 11.—Contraction of neat cements, oven-dried at 150° F

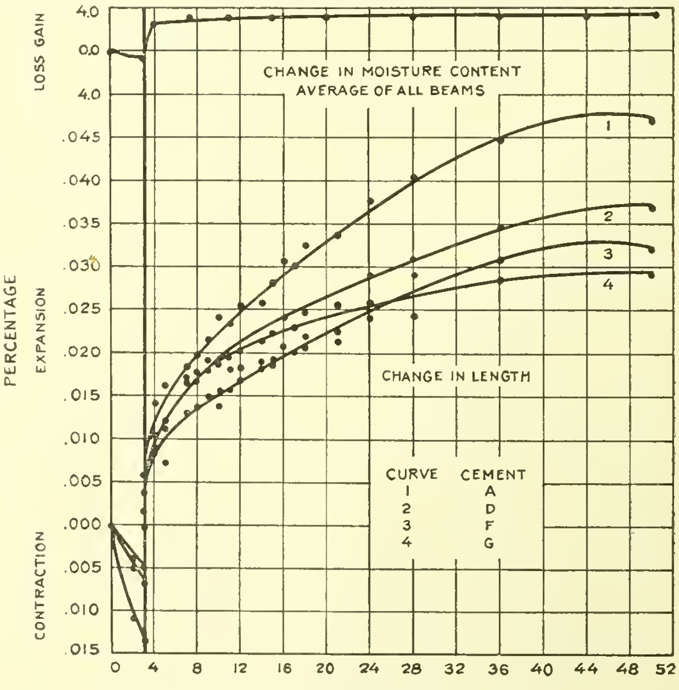


FIG. 12.—Expansion of neat cements, cured in water

The thermal coefficient of expansion of the several brands of cement tested neat was as follows:

Brand	Age 4 months; dry in air. Range 50°–73° F.	Age 2 months; under water. Range 52°–90° F.
1	0.0000096	0.0000094
2	.0000094	
3	.0000092	.0000087
4	.0000090	
5	.0000089	
6	.0000083	
7	.0000085	.0000085

Table 5.—*Expansion and contraction of neat cements and concretes*

	Expansion and contraction of brands						
	A	B	C	D	E	F	Average of A and D
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Neat cement:							
Percentage of water for normal consistency	24.00	24.00	22.00	23.00	23.00	25.00	
Percentage retained on 200-mesh sieve	17.53	22.86	20.38	23.90	16.24	19.14	
Percentage of expansion in water, 10 days	+0.056	+0.044	+0.040	+0.036	+0.034	+0.030	+0.048
Percentage of contraction in air, 90 days (subsequent to immersion)	−.052	−.062	−.080	−.094	−.114	−.142	−.073
Total percentage of change	.108	.106	.120	.130	.148	.172	.121
Net percentage of change	+.004	−.018	−.040	−.058	−.080	−.112	−.025
Concrete:							
Gravel concrete, 1 : 2 : 3½—							
Percentage of expansion in water, 40 days						+0.005	
Percentage of contraction in air, 90 days						−.035	
Percentage of change, 40 days in water, 90 days in air, 20 days in oven						−.040	
Percentage of change, 40 days in water, 90 days in air, 30 days in oven						−.050	
Gravel concrete, 1 : 1½ : 3—							
Percentage of expansion in water, 40 days							+.0050
Percentage of contraction in air, 90 days							−.0150
Percentage of change, 40 days in water, 90 days in air, 20 days in oven							−.02
Percentage of change, 40 days in water, 90 days in air, 30 days in oven							−.0225

A comparison of the basic water content and voids in the cement paste at this basic water content would no doubt throw light on these differences in the changes of length and in the quantity of water required for standard consistency and the fineness or percentage retained on the 200-mesh screen, which are also noted in Table 5. The basic water content, as defined by A. N. Talbot, past president American Society of Civil Engineers, is the percentage of water at which the voids are a minimum.

It appears that those cements which expand most in water subsequently contract least in air. The neat cement prisms expanded in water nearly ten times as much as the concrete beams; in air they contracted nearly five times as much as the concrete.

TESTS ON BEAMS

Subsidiary to the tests on the slab and designed to supplement the slab observations, tests were conducted on beams and cylinders. Two classes of beam tests were made: (1) On beams warped by nonuniform saturation; and (2) on beams uniformly saturated.

In the tests of the slab it was possible to measure the deflection and deformation of the top of the slab, but there were no means available for the measurement of bottom deformations. To provide means of estimating the bottom deformations from the measured top deformations and deflections, a study was made of similar deformations observed in the warped beams. The size of these beams, 4 inches by 7 inches by 4 feet, was chosen to give a thickness equal to that of the road slab. After initial drying the beams were placed with the bottom in contact with wet burlap and successive observations of deformation and corresponding deflections were made. Deformations were recorded at seven different depths between two pairs of specimens in each beam. The average results for three beams are shown in Figure 13, A. During nine days the lower face expanded 0.0152 per cent and the upper face shortened 0.0024 per cent. The data locate the position of the neutral axis of deformations and show the relative deformations occurring throughout the depth of the beam. The height to which the saturation, visually observed, proceeded at the end of nine days was approximately 2⅔ inches, or one-third the depth of the beams.

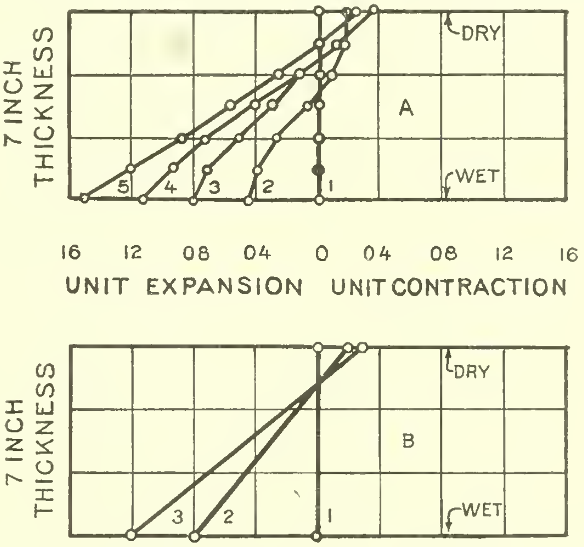


Fig. 13.—Measured deformations of beams and calculated deformations of road slab

The relative top and bottom deformations referred to above were actually observed in the test beams. To interpret the results of the slab tests, however, it is necessary to establish a relation between the top deformation and deflection and the deformation of the bottom; and it appears that a mathematical relation does exist which conforms closely with the measured values as obtained from the beams.

If the curve of distortion of the warped beams be assumed to be an arc of a circle, the following formula is applicable:

$$\epsilon_T+\epsilon_B=\frac{8\,h\,m}{L^2} \qquad (1)$$

in which—

ϵ_T = the unit deformation of the top of the beam.
ϵ_B = the unit deformation of the bottom of the beam.
h = the depth of the beam.
m = the deflection of the beam.
L = the length over which the deflection occurs.

In checking the measured deflections against the measured deformations of the beams it is found that the calculated values agree closely with the measured values.

For example, in the condition represented by curve 5, Figure 13, A, the unit expansion of the lower face of the beam was 0.0152 per cent; the unit contraction of the upper face corresponding was 0.0024 per cent; and the corresponding measured deflection was 0.0048 inch. Substituting the observed values of the unit deformations in Equation 1, the computed deflection is

$$0.000024+0.000152=0.000176=\frac{8 \times 7 \times m}{1600},$$

from which

$$m \text{ (the deflection)} = 0.0050 \text{ inch}$$

Application of formula to road slab data.—In Table 6 measured values of deformation and deflection of the top surface of the road slab are given and the corresponding bottom deformations resulting from the flooding of the subgrade are computed by substitution of the measured values in Equation 1. The curves referred to are those of Figure 13, B, and all steps in the calculation are shown.

TABLE 6.—*Computation of bottom deformations of road slab*

Curve	Condition	Measured unit contraction of top	Measured deflection over 40-inch span
		Per cent	*Inches*
1	Uniformly dry	0.02	0.0000
2	Bottom wet, 11 days (interior section)	.0019	.0028
3	Bottom wet, 11 days (exterior section)	.0028	.0042

Substituting in equation 1,

$\epsilon_T+\epsilon_B = 0.00010$ for curve 2
$.00015$ for curve 3
And, by measurement, $\epsilon_T = .000019$ for curve 2
$.000028$ for curve 3
$\therefore \epsilon_B = .00008$ for curve 2
$.00012$ for curve 3

The computed values of ϵ_B are plotted in Figure 13, B, with the measured values of ϵ_T corresponding. The figure clearly indicates the swelling of the bottom and the shortening of the top of the slab. A neutral axis of deformations separates the two, similar to that observed in the subsidiary beams. The greater swelling of the concrete at the unrestrained edge of the slab, and the lesser swelling at the interior restrained positions is shown.

The translation of the measured deformations into stresses, and the variation of such stresses throughout the cross-section, is a field of discussion of considerable interest. The corners of the slab are held up against the dead weight of the overhanging cantilever, which has been raised by the swelling of the bottom concrete. What is the state of stress in the top which has been shortened?

THE STRENGTH OF WARPED BEAMS

In addition to the above observations on warped beams, they were also used to determine the progress of saturation from the bottom upward, and the effect of the warping on their transverse strength.

TABLE 7.—*Modulus of rupture data of warped beams*

Number of beams	Initial contraction	Condition at time of test	Visual height of water	Deformation at time of test		Face loaded in tension	Modulus of rupture, cantilever loading			Modulus of rupture, third-point loading			Modulus of rupture percentage of grand average
				Wet face	Dry face		Number of values	Numerical values	Percentage of reference values	Number of values	Numerical values	Percentage of reference values	
Series No. 1. 6 inch by 8 inch by 4 foot beams. 1:1½:3 mix. Cements A and D. Age, 4 months. Oven dry.													
	Per cent		*Inches*	*Per cent*	*Per cent*								
5	0.022	Dry					20	720	100				100
2	.022	Saturated		+0.020			10	616	85				85
5	.022	Saturated to 2-inch depth	3	+.019	−0.0016	Dry.	19	789	109				109
5	.022	do	3	+.019	−.0016	Wet.	19	602	83				83
Series No. 2. 7 inch by 4 inch by 4 foot beams. 1:2:3½ mix. Cement F. Age, 3 months. Air-dry.													
3	0.041	Dry					8	839	100	3	738	100	100
3	.041	Saturated		+0.017			8	672	80	2	506	68	77
4	.041	Saturated to 2⅓-inch depth	3½	+.024	+0.0035	Dry.	12	750	89	4	614	83	87
4	.041	do	3½	+.024	+.0035	Wet.	12	618	73	4	472	64	71
Series No. 3. 7 inch by 4 inch by 4 foot beams. 1:2:3½ mix. Cement F. Age, 3 months. Oven dry.													
8	0.049	Dry					20	780	100	7	652	100	100
7	.049	Saturated		+0.018			19	671	85	5	544	83	85
3	.049	Saturated to 2½-inch depth	4	+.021	+0.016	Dry.	10	637	81	3	511	78	80
3	.049	do	4	+.021	+.016	Wet.	10	609	78	3	478	72	76
Series No. 4. 7 inch by 4 inch by 4 foot beams. 1:2:3½ mix. Cement F. Age, 7 months. Oven dry, 3 months; Air-dry, 4 months.													
1	0.037	Dry					4	809	100	1	651	100	100
1	.041	Saturated		+0.014			3	699	86	1	553	85	86
1	.043	Saturated to 1½-inch depth	3½	+.017	−0.004	Dry.	3	961	118	1	742	114	117
2	.040	Saturated to 2¾-inch depth	4	+.015	+.0003	Dry.	7	774	95	1	646	99	96
1	.040	Saturated to 4-inch depth	5	+.018	+.0017	Dry.	3	670	82	1	565	86	83
Series No. 5. 7 inch by 4 inch by 4 foot beams. 1:2:3½ mix. Cement F. Age, 7 months. Air-dry.													
2	0.051	Dry					6	825	100	2	731	100	100
1	.047	Saturated		+0.021			3	733	88	1	633	86	88
1	.048	Bottom wet	2	+.020	−0.0065	Dry.	3	1,066	129	1	884	121	127
2	.052	Saturated to 1¼-inch depth	2½	+.027	−.0048	Dry.	6	910	110	2	734	100	107
1	.054	Saturated to 2½-inch depth	3½	+.032	+.0012	Dry.	4	714	86	1	651	89	87

The beam specimens were dried to constant weight. Some were then entirely immersed in water and others were immersed to only a fraction of their depth.

The changes in weight, length, and strength were determined for the different conditions. Five separate series of such experiments were conducted, varying the proportions of the mix, the brand of cement, the extent and nature of drying, and the depth of partial saturation. The results are shown in Table 7.

Results of beam tests.—By reference to Table 7 a comparison of beams uniformly saturated shows that the expansion in water varies from 0.014 to 0.021 per cent with an average of 0.018 per cent. These beams were immersed for 5 days subsequent to the initial drying. The 1 : 2 : 3½ concrete made with cement F when air-dried for 2½ months contracted 0.041 per cent.

The 1 : 1½ : 3 concrete, made with cements A and D mixed, after conditioning by 30 days curing under wet burlap, followed by complete oven drying, contracted 0.022 per cent.

The reduction in strength from completely dried concrete to uniformly saturated concrete in the five cases ranges from 12 to 23 per cent, an average of 15.4 per cent.

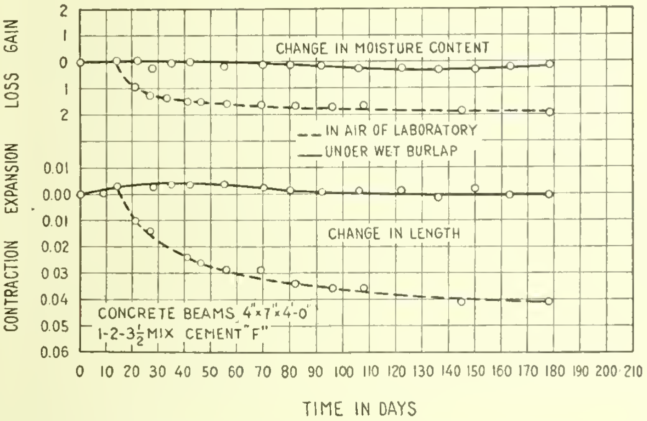


Fig. 14.—Change in length compared with change in moisture content of concrete beams made with cement F

Certain beams were warped by saturation from the bottom upward to increasing heights (see Table 7), as determined by visual observations of the broken beams and by weighing.

Contractions from 0.0016 to 0.0065 per cent were developed in the dry face, and expansions from 0.015 to 0.032 per cent at the wet face of the warped beams. When these warped beams were broken with the contracted dry side in tension, an increase from 9 to 29 per cent in strength over the entirely dry beam was indicated, depending on the amount of initial contraction.

When saturation had proceeded to a greater height in the beams (approximately 2 inches from the top), the area of dry concrete became too small to hold back the expansion of the wet area, and, consequently, the dry face was actually elongated by the force of the expanding face. When the expanded dry face was tested in tension, the beams were only 81 to 95 per cent as strong as the uniformly dry beams; when turned with the wet side in tension the percentage was 73 to 83. The complete data are given in Table 7.

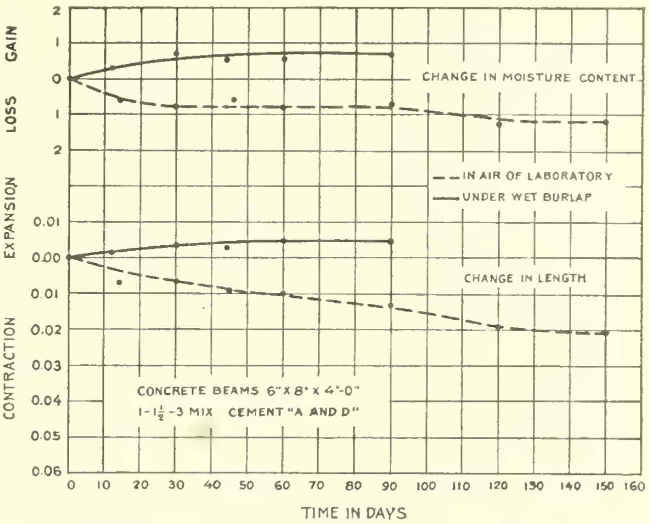


Fig. 15.—Change in length compared with change in moisture content of concrete beams made with cements A and D

The writer accounts for this increase in strength of warped dry faces when they have been shortened and the decrease in strength when they have been lengthened by the absorption of moisture on the opposite face by the presence of an initial compression in the former and of an initial tension in the latter case. However, an analysis of moduli of elasticity and the deformations of the top and bottom face of the beams, tested under the third point loading, fails to show any substantial difference between the extensibility at rupture or 50 per cent of rupture of the beams with the dry face either shortened or lengthened by warping. The neutral axis in all these loaded warped beams is nearer the compression than the tension face (wet or dry) at 50 per cent of rupture and also at rupture. The maximum extensibility in tension was from 0.015 to 0.018 per cent. The simultaneous compression was from 0.012 to 0.016 per cent.

To assume an initial stress would presuppose an elastic concrete holding this stress throughout the five days previous to test, and an unusual distribution of stresses throughout the cross section. Inasmuch as these initial stresses must balance, it is hard to reconcile an initial compression in the top with a compression in the saturated bottom. This matter is left open for discussion.

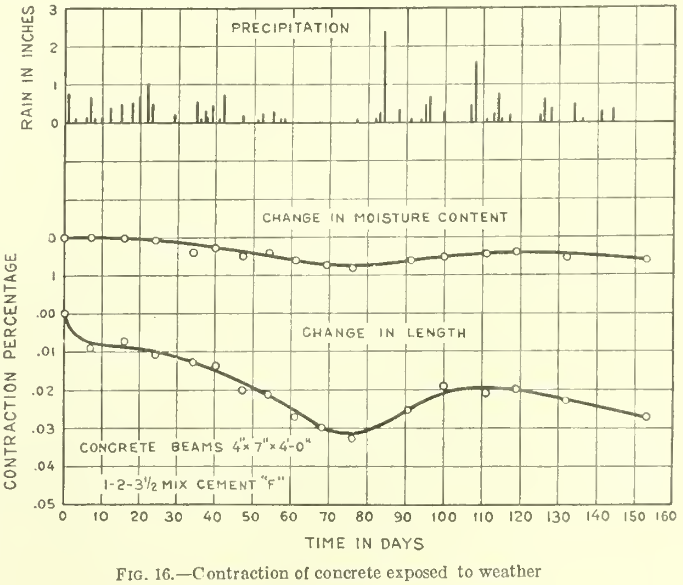


Fig. 16.—Contraction of concrete exposed to weather

TESTS OF UNIFORMLY SATURATED BEAMS AND CYLINDERS

In addition to the tests of warped beams a series of tests was made on beams and compressive cylinders with uniform moisture content. This series included the study of the following phenomena:

Length changes resulting from:
1. Continuous immersion in water.
2. Continuous exposure to air in the laboratory.
3. Continuous exposure to weather.
4. Oven drying at 150° F.
5. Immersion of oven-dried and air-dried specimens.

Transverse strength determinations as to:
1. Decrease due to uniform saturation.
2. Changes due to age and treatment.
3. Effect of freezing temperatures on saturated and dry concrete.

Compressive strength tests for:
1. Increase due to age and treatment.

The size of the beam specimens was 4 by 7 inches by 4 feet. In the initial series of tests concrete similar to that of the road slab was investigated, namely, a 1:1½:3 mix, using cements A and D. In a later series of tests, cement F was used, the mix being 1:2:3½.

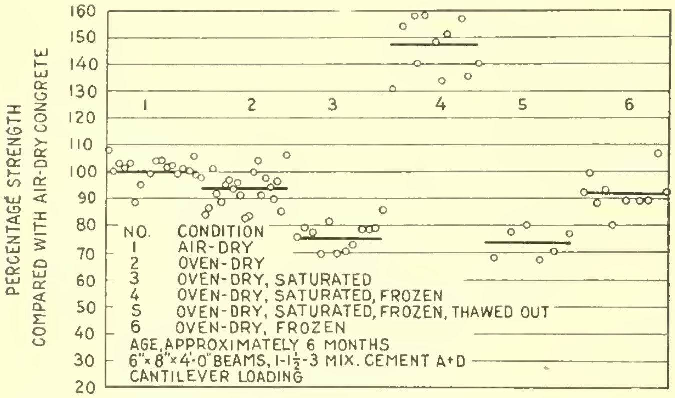


FIG. 17.—Effect of saturation and freezing on the transverse strength of beams

Length changes of beams.—Measurements of changes in length of the beams due to moisture were begun two days after moulding. A 20-inch Berry strain gauge, used in the measurements, was compared to a standard invar steel bar, maintained at a constant temperature. Six gauge lines were set in each beam, three on the top and three on the bottom. The results are the average of measurements on at least four beams. Corrections for the influence of temperature were made from observations of the temperature of two auxiliary beams, the thermal coefficient of which was previously measured.

Figures 14 and 15 show the percentage changes in moisture content and the corresponding change in unit length of the concrete. The 1:2:3½ concrete, cement F (Fig. 14), expanded 0.0050 per cent after a continuous curing under saturated burlap for 40 days and contracted 0.0035 per cent after 90 days in air. The 1:1½:3 concrete, cements A and D (Fig. 15), expanded 0.0050 per cent after a similar curing for 40 days and contracted 0.015 per cent after 90 days in air. Thus, considerable variation is evident in the degree of length change of the two concretes as affected by the character of the cement.

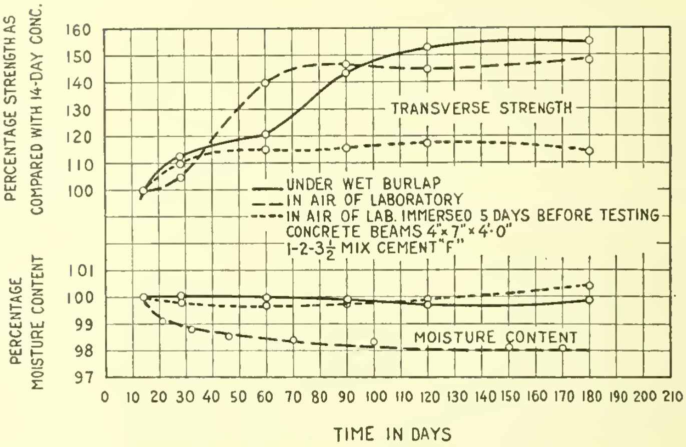


FIG. 18.—Effect of age and treatment on transverse strength of beams

Additional beams of concrete, using cement F, were exposed to the weather. Changes in length and weight and the amount of rainfall were observed, as shown in Figure 16. A maximum contraction of 0.033 per cent occurred in 75 days. Beams of similar concrete exposed to the air of the laboratory for the same period of time also contracted 0.033 per cent.

Transverse strength.—The beams were tested (1) by loading at the third points, the span length being 42 inches; and (2) by loading them as cantilevers.

In the former case, the deformations of the top and bottom faces were measured by 10-inch Berry strain gauges; in the latter, one end of the beam was fixed between the bed and the movable head of a testing machine and broken near the section of support by means of an attached lever arm 5 feet in length. In this manner a number of sections of one beam may be broken consecutively. To load the testing machine shot was allowed to flow into a bucket. The halves of the beams broken under the third-point loading were also tested under cantilever loading.

Figure 17 shows the relative strength values of a series of tests on concrete beams made with cements A and D. A 20 per cent reduction results from saturation. An increase of approximately 100 per cent in strength appears when saturated concrete is tested in a frozen condition. This is due to the strength of the ice. The percentage relation is based on air-dried concrete.

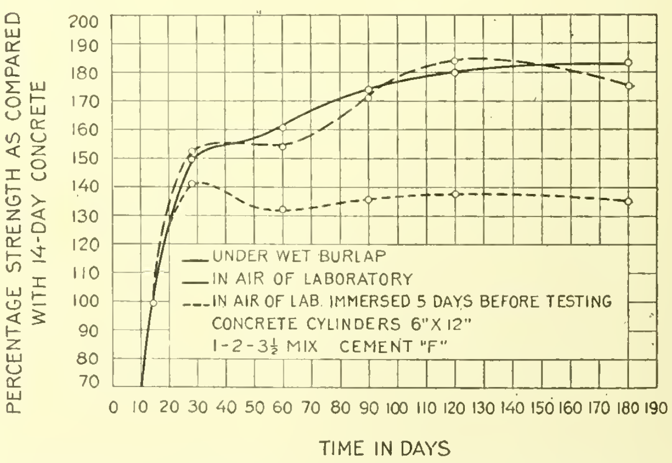


FIG. 19.—Effect of age and treatment on compressive strength of concrete cylinders

In Figure 18 the effect of age and moisture on the transverse strength of beams is shown. The specimens that were cured continuously under wet burlap and those air-dried, but subsequently saturated, contain approximately the same quantity of water by weight. The latter, however, are consistently 19 per cent lower in strength on a direct basis than the former at an age of 90 days. All the specimens were made at the same time, and then exposed to varying conditions. Each plotted point in Figure 18 represents the average of 12 tests.

No doubt the water is concentrated to some extent at the surface of the samples that are air-dried, but subsequently saturated. The appearance of the broken specimens, however, indicated uniform moisture distribution.

Compressive strength.—Cylinders 6 inches in diameter by 12 inches long were capped with plaster of Paris and tested in compression. Deformations were measured up to about one-half the ultimate strength by means of ring compressometers, with the results shown in Figure 19, which checked the findings in the case of the beams. Each plotted point represents the average of three tests.

Aggregate failures occur with consistent regularity in the case of the air-dried or oven-dried specimens, likewise with specimens cured continuously under damp burlap; in the case of the saturated concrete, however, the failures are invariably due to bond between the mortar and the aggregate.

REVIEW OF LITERATURE

A review of the literature on the expansion and contraction of cements, mortars, and concrete has been made in connection with the Purdue University investigations, and a digest appears in the paper as presented by the writer at the meeting of the highway division of the American Society of Civil Engineers, at New York, on January 22, 1925.

CURING CONCRETE WITH CALCIUM CHLORIDE

By C. L. McKESSON, Materials and Research Engineer, California Highway Commission

One of the difficulties attendant upon the construction of concrete pavements in the drier regions of California is the expense of securing water for curing the concrete for two weeks after laying. It has long been known that calcium chloride, under favorable conditions, will absorb moisture from the air and, during recent years, experiments have been made in an effort to utilize this chemical as a substitute for ordinary water curing. Illinois, with a rather high relative humidity and precipitation throughout the year, reports considerable success in the curing of concrete by distributing 2½ pounds of calcium chloride per square yard soon after the concrete is finished.

During the last year the California Highway Commission has conducted extensive field and laboratory experiments in an endeavor to ascertain whether this substitute can be used safely in California in sections where humidity is low, temperature high, and precipitation absent during summer months. Field experiments were conducted on paving jobs in Los Angeles, Ventura, Sacramento, and Humboldt Counties.

Preliminary experiments made early in the season indicated that a slight reduction in strength would probably result from the substitution of calcium chloride treatment for the usual water curing and all of the field tests made during the summer bore out this conclusion.

On two of the projects concrete specimens 6 by 6 by 12 inches were cast from the concrete actually used in the pavement. In each case half of the specimens were cured by placing them on one side the morning after they were made and by applying flake calcium chloride (dry) at the rate of 2½ pounds per square yard of surface. The other half were water cured. On three jobs, a portion of the pavement was treated in a similar way with calcium chloride and the remainder was cured with water in the usual manner. After the completion of the work cores were drilled from the pavement cured by both methods. The cores were tested by the State highway laboratory at Sacramento.

Following is a summary of the results of the tests and relative efficiency of calcium chloride curing as indicated by them.

Location of experiment	Number of samples	Average compressive strength				Efficiency of calcium chloride[1]	Remarks
		6 by 6 by 12-inch molded specimens		4½-inch cores from pavement			
		Cured with water	Cured with calcium chloride	Cured with water	Cured with calcium chloride		
				Lbs.	*Lbs.*	*P. ct.*	
(X-Sac.-4-B) Sacto-Stockton highway, near Sacto.	24	------	------	4, 292	4, 218	98	Hot and dry (water-cured concrete 57 days old. Concrete cured with calcium chloride 70 days old).
(VII-L. A.-2-BO) Ventura-Los Angeles highway near Calahasas.	32	3, 064	2, 783	------	------	91	Hot and dry.
	28	------	------	4, 057	3, 294	81	
(VII-Ventura-2-A) Los Angeles highway near Newhury Park.	12	------	------	3, 491	3, 093	88	Hot and dry.
(I-Hum.-1-GH) Eureka-Arcata highway at Arcata.	72	3, 887	3, 545	------	------	92	Experiment in October. Weather cool, humidity high (70 to 90 per cent). Some rain.

[1] Efficiency of calcium chloride curing, assuming water curing to be 100 per cent efficient.

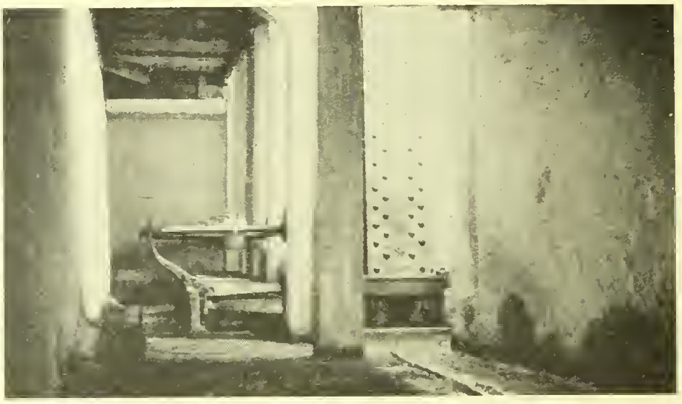
The above tests indicate that calcium chloride is from 80 to 90 per cent efficient and that it might be used as a substitute for water curing where water is scarce. Some of the strength appears to be sacrificed, but the cores showed a minimum average strength of more than 3,000 pounds in the most unfavorable case, and this strength indicates a fair factor of safety.

A very extensive series of curing tests not included in the above discussion was conducted at Sacramento during the summer and fall of 1924 by the California Highway Commission in cooperation with the Structural Materials Research Laboratory of Lewis Institute. Calcium chloride and many other methods of curing were included in this series, the results of which will be given to the public in a report now being prepared.

VERTICAL PRESSURE OF EARTH FILLS MEASURED[1]

By C. N. CONNER, State Construction Engineer, North Carolina State Highway Department

Clay fill at 13 foot level



Scales in place in culvert

VALUABLE data on the vertical pressure of earth fills of various materials are expected to result from experiments now being conducted by the University of North Carolina in cooperation with the North Carolina State Highway Commission.

The curves in Figure 1 show the results of some of the early experiments with sand and clay fills. It is interesting to note that greater pressures are brought to bear on the 30-inch cast-iron pipe by the clay fill than the sand fill at all depths.

The experiments are being conducted under the direction of G. M. Braune, dean of the School of Engineering, University of North Carolina, and P. K. Schuyler, associate professor of highway engineering, is actively supervising the work. The plans for the weighing scales and the general methods in use were suggested and approved by A. T. Goldbeck, chief, division of tests, United States Bureau of Public Roads.

The methods used to determine the pressures are as follows: Four 2½-foot sections of 30-inch cast-iron pipe were placed on columns, which rested on the weighing platforms of specially designed scales. The scales and columns were housed in a concrete culvert in the top of which there was a slot just large enough to admit the pipe, which lay in a horizontal position, with one half of its surface protuding above the top of the culvert and exposed to the pressure of the earth fill, while the other half of its surface was inside the culvert resting on the top of the column.

A sand fill, using uniform quality of sand, was built up in 1-foot layers, each layer being carefully leveled off and the pressures accurately determined by weighing the loads on the various scales. An attempt was made to roll the sand fill, but it was not found to be practicable. The experimental fill with sand was carried to a height of 20 feet above the top of the pipe. The sand fill was then removed and replaced with a clay fill. This fill was also placed in 1-foot layers and readings taken at each foot level in a manner similar to the method employed with the sand fill. The clay fill was rolled with a 4-ton roller and carried to a height of 14 feet. The next part of the experiment as proposed will consist of using a clay fill and laying the pipe in a trench excavated after the fill is made. It is believed that by laying a pipe in this manner the pressure exerted on it by the fill is lessened.

Maj. William Cain, an authority on earth pressure, has suggested certain laboratory experiments, which are being carried out in connection with the field work. It is thus hoped to obtain data on the coefficients of friction and cohesion of soils, which can be applied to the results obtained in the field and thereby add to the present available information on the subject of earth pressure.

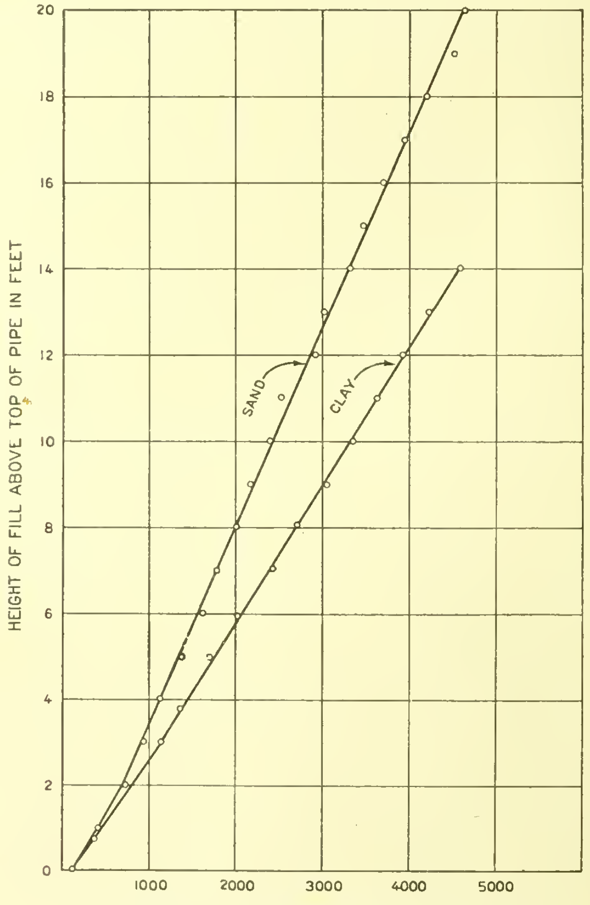


FIG. 1.—Pressures produced by sand and clay fills on 30-inch cast-iron pipe

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[1] Submitted for publication through the Highway Research Board of the National Research Council.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

FEBRUARY 28, 1925

GOVERNMENT PRINTING OFFICE

STATES	FISCAL YEARS 1917-1924			FISCAL YEAR 1925									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1924			PROJECTS COMPLETED SINCE JUNE 30, 1924			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 4,598,721.63	$ 2,186,247.54	464.1	$ 1,211,826.12	$ 597,175.34	132.2	$ 14,988,146.68	$ 7,263,473.51	802.8	$ 79,416.15	$ 39,708.07	0.4	$ 2,708,228.54	Alabama
Arizona	8,338,366.41	4,287,683.88	527.8	713,173.87	406,825.70	37.5	1,985,482.12	1,203,724.85	169.2	666,410.72	407,243.56	61.6	2,246,394.01	Arizona
Arkansas	11,094,751.31	4,424,345.63	944.4	1,637,568.79	688,383.74	68.1	7,077,064.30	2,975,484.70	382.0	390,704.48	154,050.66	22.2	2,084,299.27	Arkansas
California	12,999,075.03	5,647,149.17	533.7	6,134,843.27	3,307,620.89	257.3	10,534,164.28	5,239,754.58	276.9	14,898.29	8,789.99		5,362,628.37	California
Colorado	8,108,070.31	4,029,998.97	502.6	1,578,787.09	829,849.40	71.3	4,982,544.65	2,688,815.09	202.4	476,373.92	177,583.34	11.1	3,206,971.20	Colorado
Connecticut	3,062,872.02	1,269,559.60	73.6	199,024.56	98,423.00	4.9	3,262,054.48	1,045,804.04	54.1				1,442,210.36	Connecticut
Delaware	3,056,832.22	1,007,714.83	72.5	951,730.50	380,775.82	27.4	785,048.44	321,281.10	19.5				396,383.25	Delaware
Florida	961,134.07	461,470.92	48.8	1,840,381.57	894,017.05	40.4	8,206,935.44	3,971,175.98	231.9	506,722.44	190,336.40	11.5	1,662,764.65	Florida
Georgia	17,167,373.32	7,965,805.20	1,214.2	1,890,830.71	916,025.00	169.6	11,397,917.44	5,350,291.47	777.0	367,074.37	128,236.68	34.9	2,082,627.65	Georgia
Idaho	8,181,697.92	4,092,395.52	506.8	1,025,866.02	584,588.92	68.4	1,883,697.06	1,160,646.54	129.9	590,256.73	397,532.32	43.9	1,379,475.70	Idaho
Illinois	26,964,706.06	12,279,546.33	804.7	4,724,921.20	2,320,686.26	152.4	12,467,692.53	6,188,330.64	428.8				5,839,407.77	Illinois
Indiana	7,577,444.16	3,655,540.97	225.7	3,847,801.03	1,846,705.95	130.6	13,309,694.44	6,613,176.10	428.3				4,235,661.98	Indiana
Iowa	23,195,778.19	9,237,031.86	1,682.9	2,995,755.18	1,367,916.81	228.1	6,154,529.09	2,793,411.59	439.9	2,862,077.21	1,202,126.05	105.3	2,806,046.69	Iowa
Kansas	17,084,136.48	6,043,176.80	502.7	6,634,089.23	2,611,801.08	228.5	13,887,101.93	5,650,610.41	576.0	2,476,897.02	976,253.63	120.5	2,091,807.08	Kansas
Kentucky	10,822,980.31	4,613,947.28	429.4	2,635,711.90	1,052,626.11	99.2	8,028,739.26	3,646,720.61	298.9	406,326.77	140,280.00		2,329,772.10	Kentucky
Louisiana	8,488,463.18	3,636,143.36	661.2	1,796,469.94	876,444.77	151.8	4,508,500.61	2,245,730.04	260.4	118,433.78	59,216.89	0.5	1,446,168.94	Louisiana
Maine	6,911,058.78	3,299,935.38	230.7	1,263,222.53	607,934.95	50.7	749,878.17	350,363.39	24.2				1,516,878.28	Maine
Maryland	6,760,044.42	3,213,321.78	243.2	646,410.32	307,486.47	25.5	2,464,768.03	1,065,404.17	76.2	166,738.11	55,650.00	4.2	642,870.58	Maryland
Massachusetts	10,191,202.02	4,105,727.22	232.8	566,949.13	184,277.95	9.8	6,131,750.58	1,967,428.71	100.2	1,519,948.77	426,439.71	25.4	2,326,024.41	Massachusetts
Michigan	13,434,135.07	6,060,612.23	494.5	2,030,127.09	928,663.92	85.8	14,828,927.22	7,004,292.58	491.8	163,170.00	81,685.00	5.4	4,029,845.27	Michigan
Minnesota	24,037,561.24	9,885,843.07	2,292.0	6,378,124.65	2,852,798.97	429.2	6,294,034.56	2,526,600.00	650.8	679,082.36	46,300.00	67.8	2,131,027.96	Minnesota
Mississippi	7,888,193.89	3,828,941.39	655.0	1,840,746.14	881,221.43	136.4	8,318,899.13	4,160,193.92	481.1	205,237.88	102,618.94	13.3	1,830,257.32	Mississippi
Missouri	11,352,027.70	5,245,899.18	803.5	3,418,813.73	1,691,588.82	186.6	23,530,361.95	9,699,116.19	805.9	4,475,883.97	897,994.12	166.7	2,823,316.69	Missouri
Montana	8,867,279.16	4,384,335.12	791.4	1,240,143.42	907,104.11	127.3	1,935,343.45	1,406,802.11	164.3	1,038,200.22	590,707.66	117.2	5,225,941.00	Montana
Nebraska	7,876,337.16	3,714,691.59	1,440.4	552,461.12	263,623.02	32.0	8,329,203.67	4,082,797.91	902.5	365,085.32	182,542.65	35.2	4,789,259.83	Nebraska
Nevada	3,460,246.52	1,853,624.98	225.6	1,446,657.71	1,228,225.07	131.4	4,300,066.51	3,622,200.69	361.5				1,134,346.26	Nevada
New Hampshire	3,076,750.19	1,487,867.58	171.3	976,255.13	453,342.53	35.0	855,015.12	401,745.62	25.8	40,306.73	19,027.23	1.6	438,606.04	New Hampshire
New Jersey	7,623,795.12	2,661,531.49	148.7	3,474,504.68	921,087.99	55.4	7,297,701.96	2,277,518.25	40.1				1,664,191.27	New Jersey
New Mexico	5,306,286.45	2,758,849.68	714.3	1,897,320.22	1,216,549.20	196.8	5,677,646.65	3,719,421.07	554.5	250,983.39	154,354.77	16.7	1,925,323.28	New Mexico
New York	18,862,742.49	8,257,844.44	572.7	2,048,366.93	841,486.31	57.9	30,373,274.84	11,391,189.38	676.0	5,738,088.00	1,690,100.00	109.7	8,184,623.87	New York
North Carolina	12,567,732.97	6,676,757.66	884.7	5,596,875.90	2,019,346.21	156.7	6,998,334.85	2,816,788.84	194.6	2,077,691.35	925,978.97	59.6	2,554,547.32	North Carolina
North Dakota	9,088,973.11	4,418,505.42	1,587.9	1,538,610.56	751,591.33	304.8	2,668,572.13	1,339,895.23	367.9	390,407.61	195,203.76	33.6	2,839,159.26	North Dakota
Ohio	33,122,751.43	11,879,917.99	962.5	7,315,479.45	2,996,275.94	198.2	8,676,542.09	3,008,307.50	253.1	2,461,465.64	924,794.15	72.3	4,120,456.42	Ohio
Oklahoma	12,986,865.26	5,888,852.03	497.3	5,198,375.46	2,515,773.32	216.9	6,819,841.25	3,181,417.71	325.0	1,072,902.56	446,501.55	70.7	2,259,263.39	Oklahoma
Oregon	12,082,873.17	5,819,093.79	655.6	1,209,085.39	707,463.39	95.4	3,385,451.57	1,963,099.34	150.5				1,196,170.48	Oregon
Pennsylvania	36,826,248.98	14,114,694.79	729.7	2,621,735.82	1,034,703.38	56.5	22,289,117.00	5,881,352.50	364.1	9,699,397.38	3,392,560.00	225.8	3,538,428.33	Pennsylvania
Rhode Island	1,774,397.25	779,227.96	46.0	854,098.95	340,460.13	18.8	1,171,551.79	341,168.84	15.9	248,821.10	86,175.00	5.7	761,634.07	Rhode Island
South Carolina	9,016,476.73	4,124,045.22	924.4	1,751,203.47	810,291.65	269.6	6,165,301.24	2,330,703.22	402.5	463,013.22	112,074.48	9.4	1,362,980.43	South Carolina
South Dakota	8,674,597.86	4,244,636.27	989.8	2,807,897.50	1,465,555.64	424.5	6,222,192.74	2,937,305.56	803.5	220,684.20	4,000.00	45.5	1,282,202.53	South Dakota
Tennessee	6,805,683.36	3,313,936.07	259.6	5,564,284.08	2,752,118.58	192.5	11,702,655.73	5,293,372.60	422.4	677,750.73	338,876.36	37.1	1,949,319.39	Tennessee
Texas	42,341,998.56	16,190,624.91	3,122.8	7,749,029.67	3,130,067.52	505.0	23,982,053.14	9,991,360.29	1,462.8	3,895,293.86	1,792,562.98	254.3	5,036,322.30	Texas
Utah	3,304,423.75	1,895,805.92	219.0	1,379,283.77	875,315.41	130.5	4,144,168.50	2,721,727.32	242.3	339,709.28	236,839.03	31.5	1,233,252.32	Utah
Vermont	1,922,114.16	942,769.12	74.4	347,337.97	162,957.67	9.6	1,939,138.59	929,470.01	49.6	10,699.53	5,349.76		859,057.44	Vermont
Virginia	10,035,301.48	4,801,782.43	562.5	1,670,046.39	792,999.52	67.7	10,109,396.39	4,667,709.01	346.6	759,458.11	315,078.77	31.0	1,465,096.27	Virginia
Washington	11,394,615.67	5,290,995.45	467.0	1,172,750.16	511,294.59	51.9	3,367,073.68	1,599,900.00	134.9	296,296.71	134,000.00	9.1	1,469,574.96	Washington
West Virginia	5,489,747.95	2,365,041.53	255.6	1,853,452.91	865,251.90	71.1	4,166,291.65	1,791,537.78	127.9	125,835.05	51,816.75	5.1	1,477,780.14	West Virginia
Wisconsin	18,753,903.15	7,441,033.57	1,325.3	2,196,436.45	1,064,453.50	92.5	3,175,101.72	1,516,953.95	139.2	53,417.25	26,708.00	3.9	5,512,709.98	Wisconsin
Wyoming	6,127,625.61	3,078,098.70	687.6	2,253,839.73	1,385,969.97	254.9	3,140,910.64	1,959,362.40	157.9	96,301.49	55,793.00	8.4	1,143,073.93	Wyoming
Hawaii													731,250.00	Hawaii
TOTALS	549,655,391.27	237,952,399.92	32,452.9	120,677,497.41	56,237,136.23	6,544.4	374,668,979.28	166,224,937.14	16,731.4	46,476,360.70	17,172,998.23	1867.1	120,762,638.58	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 113,536,994.31 Federal aid $ 50,031,036.03 Miles 4,446.9

MOTOR VEHICLE REGISTRATION AND REVENUES, REGISTRATION YEAR 1924

State	Individually and commercially owned				Official cars and trucks owned by State, etc.[1]	Motorcycles	Registration fees, licenses, permits, etc.		Amount of registration fees paid for		Grand total motor cars 1923	Per cent increase during 1924	State
	Grand total motor cars 1924[1]	Passenger cars[1]	Motor trucks[1]	Taxis, busses, and cars for hire			Total gross receipts	Amount applicable to highway work by or under supervision of State highway department	Passenger cars	Motor trucks			
												Per cent	
Alabama	157,262	135,777	18,688	2,797	(2)	549	$1,954,801	$1,581,047			126,642	24.2	Alabama.
Arizona	57,828	50,233	7,595	(3)	903	372	339,722	339,722			49,175	17.6	Arizona.
Arkansas	141,983	125,368	16,615	(3)	458	295	2,333,210	1,833,240	$1,980,814	$262,317	113,300	25.3	Arkansas.
California	1,319,394	1,125,381	[4] 191,013	(4)	(3)	12,325	7,011,113	3,079,659	3,594,636	2,440,377	1,100,283	19.9	California.
Colorado	213,247	197,361	15,886	(5)	(2)	2,226	1,258,205	574,568	992,333	180,222	188,956	12.9	Colorado.
Connecticut	217,227	180,542	33,776	2,909	1,110	4,211	5,069,581	5,069,581	2,766,530	1,047,278	181,748	19.5	Connecticut.
Delaware	35,136	29,075	6,061	(3)	(3)	325	604,354	604,354	334,250	122,874	29,977	17.2	Delaware.
District of Columbia	88,762	78,846	9,916	(5)	1,351	1,889	378,868				[6] 74,811	18.6	District of Columbia.
Florida	195,128	157,519	34,192	3,417	(3)	733	2,418,933	1,576,118			151,990	28.4	Florida.
Georgia	207,688	181,268	26,275	115	(3)	750	2,532,266	2,446,215	2,067,280	408,823	173,889	19.4	Georgia.
Idaho	69,227	61,600	7,627	(3)	930	619	1,306,892	326,723	1,083,700	195,727	62,379	11.0	Idaho.
Illinois	1,119,236	978,428	140,808	(3)	(3)	6,873	11,546,206	11,546,206			969,331	15.5	Illinois.
Indiana	651,705	566,736	84,969	(3)	(3)	4,822	4,102,666	3,906,858	3,030,023	826,008	583,342	11.7	Indiana.
Iowa	616,128	575,210	40,918	(5)	2,400	2,597	8,979,170	[7] 8,171,045			571,061	7.9	Iowa.
Kansas	410,891	370,951	39,940	(2)	1,947	1,632	[7] 4,222,930	[7] 4,036,937			375,594	9.4	Kansas.
Kentucky	229,804	206,064	23,275	[8] 465	1,044	724	3,233,379	3,108,732			198,377	15.8	Kentucky
Louisiana	[7] 178,000	[7] 150,900	[7] 27,100	(5)	1,000	510	2,790,348	2,790,348	[7] 1,922,716	[7] 813,000	136,622	30.3	Louisiana.
Maine	127,178	105,040	19,001	3,137	854	1,288	1,933,561	1,839,259			108,609	17.1	Maine.
Maryland	198,398	184,398	11,183	2,817	(3)	3,462	2,332,953	1,633,067	1,399,020	247,295	169,351	17.2	Maryland.
Massachusetts	[1] 570,578	[1] 485,952	[1] 83,626	(5)	[9] 900	10,778	8,122,166	[7] 7,400,090	5,119,148	1,233,626	[1] 479,909	18.9	Massachusetts.
Michigan	867,545	784,070	83,475	(3)	(3)	3,644	12,404,546	5,638,050	9,730,255	1,408,579	730,658	18.7	Michigan.
Minnesota	503,437	465,165	37,823	449	2,171	3,080	8,591,853	8,591,853	7,387,698	886,036	448,187	12.3	Minnesota.
Mississippi	134,680	122,117	12,563	(5)	(2)	96	1,525,077	589,844			104,286	29.1	Mississippi.
Missouri	540,500	489,356	51,144	(5)	1,203	2,139	4,525,914	4,238,914			476,598	13.4	Missouri.
Montana	79,695	69,824	9,871	(3)	979	293	776,320		638,534	107,310	73,828	7.9	Montana.
Nebraska	308,715	277,449	31,266	(2)	(3)	1,342	3,597,261	[7] 2,697,946	2,922,756	564,702	286,053	7.9	Nebraska.
Nevada	18,118	16,236	1,882	(5)	336	111	181,970	172,000	142,528	34,168	15,699	15.4	Nevada.
New Hampshire	70,932	63,662	7,270	(3)	(2)	1,750	1,522,186	1,411,794			59,604	19.0	New Hampshire.
New Jersey	504,217	393,785	99,288	11,144	[7] 4,200	8,053	9,278,428	8,213,182	3,673,989	2,701,805	430,958	17.0	New Jersey.
New Mexico	41,680	39,890	1,790	(3)	(3)	228	421,412	400,342			32,032	30.1	New Mexico.
New York	1,412,879	1,136,678	236,012	40,189	8,910	19,837	24,089,241	18,066,930	14,001,939	6,235,099	1,204,213	17.3	New York.
North Carolina	302,232	272,552	27,480	2,200	(3)	1,029	4,614,521	4,153,069			[10] 246,812	22.5	North Carolina.
North Dakota	117,346	112,664	4,682	(3)	[9] 317	509	816,766	[11] 773,691			109,266	7.4	North Dakota.
Ohio	1,241,600	1,076,800	164,800	(3)	5,400	15,000	11,685,329	5,842,664			1,069,100	16.1	Ohio.
Oklahoma	369,903	342,856	27,047	(3)	(3)	733	3,728,679	[12] 3,323,609			307,000	20.5	Oklahoma.
Oregon	192,615	177,558	15,057	(3)	(3)	2,764	4,766,070	3,424,552	3,925,444	688,712	165,962	16.1	Oregon.
Pennsylvania	1,228,587	1,043,692	178,122	6,773	(3)	17,540	22,107,376	22,107,376	10,236,151	4,870,202	1,043,770	17.7	Pennsylvania.
Rhode Island	95,482	76,666	17,247	1,569	(3)	1,428	1,623,604	[7] 1,523,604	935,586	370,886	76,312	25.1	Rhode Island.
South Carolina	161,753	146,639	15,114	(3)	1,067	477	1,151,983	921,586	933,163	192,154	127,467	26.9	South Carolina.
South Dakota	142,396	131,190	11,206	(3)	(3)	305	2,068,437	1,445,920			131,700	8.1	South Dakota.
Tennessee	204,680	183,891	20,789	(3)	(3)	682	2,597,870	2,597,870	2,021,931	534,079	173,365	18.1	Tennessee.
Texas	801,712	735,270	62,754	3,688	(2)	2,634	10,373,997	7,225,991			688,233	16.5	Texas.
Utah	68,316	59,453	8,863	(3)	500	731	485,969	427,509	379,972	87,992	59,525	14.8	Utah.
Vermont	[13] 61,179	[13] 57,072	[13] 4,107	(3)	123	779	1,323,377	1,252,101	1,008,165	101,452	52,776	15.9	Vermont.
Virginia	261,945	220,000	41,643	302	(3)	3,000	3,791,556	3,791,556			218,896	19.7	Virginia.
Washington	295,443	251,466	41,555	2,422	3,701	3,164	4,861,420	4,416,053	3,230,688	950,127	258,264	14.4	Washington.
West Virginia	190,734	163,907	22,171	4,656	(3)	1,407	2,874,587	2,332,712	1,949,982	449,016	157,924	20.8	West Virginia.
Wisconsin	525,221	475,182	50,039	(3)	3,005	3,938	6,786,485	[7] 6,500,000	5,483,275	1,160,967	457,271	14.8	Wisconsin.
Wyoming	43,639	38,831	4,808	(3)	177	252	448,664	448,664	346,365	90,622	39,831	9.6	Wyoming.
Total	17,591,981	15,371,570	2,131,332	89,079	44,986	153,925	225,492,252	184,393,071	93,269,171	29,211,455	15,090,936	16.6	Total.

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[1] Net number of cars and trucks shown when possible, excluding reregistrations and nonresident registrations. Federal, State, or other Government owned cars and trucks, not registered and not paying licenses, are also excluded in grand totals, unless noted.

[2] Recorded in private cars and trucks.

[3] Not separately recorded.

[4] "Motor trucks" includes solid and pneumatic types, also taxis, busses, etc.

[8] City cabs excluded.

[9] State owned cars only.

[10] First six months of registration year only.

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 6, NO. 2 ▽ APRIL, 1925

A TYPICAL ELEVATING GRADER-WAGON OUTFIT

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THE WAGON AND THE ELEVATING GRADER

AN ECONOMIC STUDY OF THE WAGON-ELEVATING GRADER COMBINATION IN THREE PARTS

By J. L. HARRISON, Highway Engineer, U. S. Bureau of Public Roads

PART I

This is the fourth of a series of articles on the factors governing construction costs in the highway field. Other articles will be published from time to time. The Bureau of Public Roads desires that the material presented in these articles shall be as helpful as possible to engineers and to contractors. To that end discussion is cordially invited, either through the engineering press or through this magazine, and any contributions in amplification or in criticism of the facts or the deductions published, if tending to clarify the information presented, to broaden the scope of this work, or to correct deductions thought to be erroneous, will be given appropriate consideration.

IN ANY consideration of the operation of the wagon-elevating grader outfit it is well to remember that the elevating grader is a digging and loading mechanism, and that the wagon is a hauling mechanism. In such an outfit the two styles of equipment perform wholly different functions. In this respect the elevating grader outfit differs somewhat from the older forms of dirt-moving apparatus. Fresnoes and slip scrapers load with the same power that is used in hauling. The wheel scraper requires some additional power for loading—the snatch team—but uses no separate loading device. But in the wagon-elevating grader outfit the grader is primarily a loading device and the production of the outfit as a whole is governed by this apparatus, for, obviously, no more material can be moved by the wagons than can be taken out and loaded by the grader.

The elevating grader was devised for one purpose—to load wagons quickly and economically. The bottom-dump wagon is a much more efficient hauling device than either slips, fresnoes, or wheel scrapers. An ordinary slip, if well loaded, moves about one-fifth of a cubic yard per load, a fresno about one-third of a cubic yard, and a No. 2 wheeler about two-fifths of a cubic yard per load. To move these relatively small amounts, two horses and one man are required in the operation either of a slip or a wheeler while it is engaged in haul. The commonly used 4-foot fresno employs one man and three horses. As compared with this, a standard 1½-yard, bottom-dump wagon hauled by two good horses and in charge of one driver will carry from 1.2 to 1.4 cubic yards, and a 2-yard bottom-dump wagon with a driver and three horses can be loaded to its full rated capacity of 2 cubic yards. These quantities are net, that is, as measured in place, and while they are not always secured under ordinary operating conditions, a failure to secure them is generally due to indifferent supervision of the loading. But, to make effective use of this large advantage in carrying capacity, it is essential that a wagon be loaded quickly and economically. This is the function of the elevating grader and the success of the wagon-elevating grader combination depends on its attainment.

THE elevating grader is an excavating and loading machine depending upon wagons for hauling the material which it excavates. In this respect it differs from other types of grading equipment, such as slip scrapers, fresnoes, and wheel scrapers, which haul as well as excavate the earth.

In presenting this study of the operation of the elevating grader-wagon combination the discussion has been divided into three parts; the first of which, dealing with the operation of the grader and the various factors affecting its production, is published in this issue.

Theoretically the capacity of an elevating grader is about 5,000 cubic yards per 10-hour day. Yet many contractors obtain as little as 700 and few attain an average daily production of 1,000 cubic yards. This wide difference between the theoretical and actual production is due to a number of losses, some of which are inherent in the machine itself or the working conditions and, therefore, are unavoidable. Others, however, can be eliminated by careful study and supervision. It is the purpose of this part of the article to explain the nature of these various losses and indicate what may be done to avoid them.

The theoretical capacity of 5,000 cubic yards would be obtained if the grader operated continuously at a speed of 3¾ feet per second without loss of time for a 10-hour day taking constantly a bite with a cross section of 1 square foot and loading all excavated material.

In average practice failure to take a full bite reduces the capacity at once to 2,500 cubic yards. In part this is due to the inefficiency of the bull wheel which transmits power to the plow; in part it is due to the necessity of cutting to a level cross section which, when the original ground level slopes transversely, requires that a shallower bite be taken on the low side of the cut.

Failure to operate at the possible speed of 3¾ feet per second also reduces the quantity of material excavated; and losses of excavated material back of the disk plow and over the sides of the elevating belt still further reduce the material loaded. The fact that the grader does not excavate when turning at the two ends of the cutting loop and time losses in exchanging wagons also subtract from the theoretical capacity.

The wagon-exchange losses are in large part avoidable by proper adjustment of the wagon supply to the length of haul and by requiring the drivers to approach the grader in close order. With lax supervision the time required for the exchange is sometimes as high as 25 seconds. It is generally not less than 11 seconds; but it can reasonably be reduced to 5 seconds.

Inefficient maintenance of the grader resulting in frequent breakdowns, and time lost in cleaning the belt and resting stock still further reduce the production; and these losses especially the first and third, are largely avoidable.

The conclusion drawn from a careful analysis of all losses is that average production can be readily increased from 700 cubic yards a day to at least 1,500 without change in the present design of the grader. By suitable modification of the machine along lines suggested a still greater increase would result.

As a digging and loading tool the elevating grader has its peculiar field, which is the handling of clay and loam in level to rather heavily rolling country. It is not well adapted to work in rugged country where the slopes are steep and changes in slope abrupt, and, if the existing cuts are deep, it can not be used to advantage where the work consists largely of widening old roads. Neither is it well adapted, as now commonly designed, for handling sand or for working over wet ground. Its best field is excavation on new location, particularly in those regions, as the Mississippi Valley, where the soils are deep and rock and bowlders are seldom encountered. It does not operate satisfactorily in cut-over land unless the grubbing is so thoroughly done that the larger roots are removed.

THE ELEVATING GRADER

In principle, the machine consists of a plow (generally a disk plow) so mounted on a heavy frame, which in turn is mounted on four wheels, that the spoil is dropped on a moving belt by which it is elevated to a height from which it can be dropped into a bottom-dump wagon. Power for driving the belt is picked up by a bull wheel [1] as the apparatus is hauled over the ground either by the 16 to 20 horses required to operate it [2] or by a large tractor. The simple operation of raising the dirt from the ground to the height from which it must be dumped into the wagon requires from 3 to 6 horsepower. As a mechanical device, any bull wheel is highly inefficient; so much so, in fact, that, installed on the elevating grader, it must pick up power at the rate of from 5 to 15 horsepower in order to supply the power needed to elevate the material produced by the plow. This matter is mentioned because it shows why this machine does not function properly in loose, dry sand and in wet ground, its lack of efficiency in such materials being due to the fact that they do not offer enough tractive resistance to enable the bull wheel to supply the power required in operating the belt. Doubtless, as the design of these machines is improved, the models in common use will be provided either with independent power plants for operating the elevating mechanism or, as in the case of the modern harvester, with connections that will make it possible to secure power by direct transfer from the prime mover.

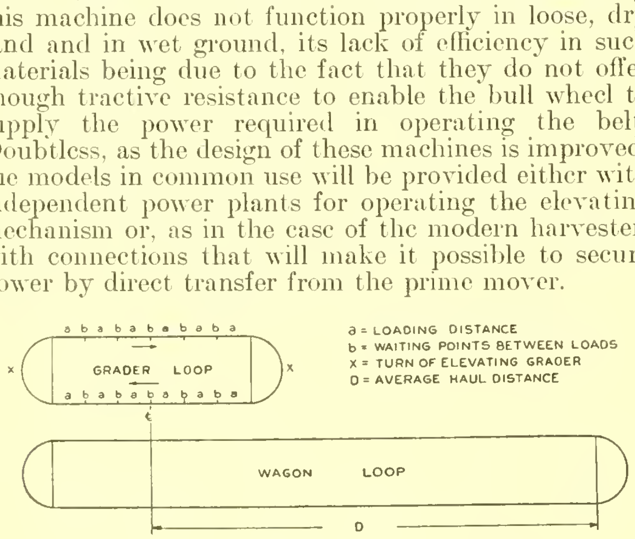

FIG. 1.—Diagram of elevating grader and wagon operation

As the engineer who is estimating a prospective project or a contractor who is working on a project with an outfit of this character, must view his elevating grader as an excavating machine and his wagon supply as a means of hauling the material excavated, his problem is automatically divided into three parts: (1) The production that his grader can secure; (2) the distance the earth must be hauled; and, (3), the wagon supply which must be provided in order to move this production over this distance.

OPERATION OF THE GRADER

As a basis for studying the production of the elevating grader, it may be noted that it is always worked in a loop, i. e., it is hauled back and forth over the area from which material is being taken. This loop is composed of a cut down one side of the area, a turn at the end of the cut, a cut up the other side, and another turn. (See Fig. 1.) Occasionally the lay of the work permits cutting on only one side of the loop; and no material is ever taken out on the turns.

While working along the sides of the loop the operating cycle consists of a period during which the elevating grader is moved forward taking out material, elevating it and depositing it in a wagon which moves alongside, followed by a period during which the grader stops while the loaded wagon moves forward allowing another wagon to move in and take its place. The cutting operation when loading wagons is, therefore, a series of moves forward followed by periods of rest. When casting, no wagons are used and as a consequence the cutting operation between turns is continuous. The speed at which the grader is hauled varies somewhat as between different outfits, but for reasons that will appear later, should be at least 3¾ feet per second.

The cross section of the cut taken out by the plow or disk also varies but, under favorable conditions, may be as high as 1 square foot. Table 1 shows typical stop-watch readings on loading time and miscellaneous data on a project where soil conditions were favorable. A load of 1.35 cubic yards is about 36.5 cubic feet. As, during this study, the grader was being hauled at the rate of 3.62 feet per second, slightly over 0.9 cubic foot of material was delivered to the wagons for each foot of run whenever the loading time dropped to 11 seconds. Allowing for losses of material back of the disk, along the belt, and off the wagons, losses that in good loam are considerable, it is apparent that during this study the cross section at the disk was not infrequently reaching a square foot. This rate of loading is not uncommon if soil conditions are favorable, and may occasionally be slightly exceeded. However, it is not a common thing to find that it is maintained as an average for any considerable time or that it prevails during any large part of the work. It is rather to be viewed as a maximum and therefore as a convenient basis from which to calculate performance.

THEORETICAL CAPACITY OF THE ELEVATING GRADER

The amount that an elevating grader will deliver depends then on a number of things, but it will be apparent that if the cross section of the bite could be consistently maintained at 1 square foot and the speed at 3¾ feet per second, the output of the machine would reach the astonishing rate of 8.3 cubic yards per minute, or about 5,000 cubic yards per 10-hour day, which may be considered as the theoretical capacity of the machine. As many contractors secure a production of as little as 700 cubic yards per day and few indeed attain a daily average of 1,000 cubic yards, there is a wide difference between the theory, as here set down, and actual practice. This difference is caused by losses of one sort or another, and the very magnitude of their cumulative effect suggests their study in some detail.

It will be obvious, almost at a glance, that these losses must fall under one of three headings: (1) Failure to maintain the full cross section while cutting; (2) failure to maintain full speed while moving; and (3) time losses during the working period.

It may be stated categorically that the full cross section of the bite can not be maintained in highway excavation. There are a number of reasons for this. In the first place the cross section of the bite is affected by the soil in which the apparatus is working. In light soil, if the traction is good, the cross section may slightly exceed a square foot. If the soil is heavy and comes off the disk in strips or ribbons, but the traction is good, an almost equally high rate of cutting may be had. But if the soil is such that it tends to clog the belt or if the traction is such that there is more or less slipping of the bull wheel—a very common condition—the cross section must be reduced to a point where the power that can be picked up by the bull wheel is sufficient to elevate the material produced. There is also the fact that in many soils, particularly moist clay, the belt fouls badly and as a result tends to slip whenever the load is heavy. This condition works toward a reduction of the bite as whenever it exists the load delivered to the belt must be reduced to a point where the belt will function satisfactorily.

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[1] Designs equipped with an independent power plant for the operation of the belt have been on the market for some years but are not in common use.

[2] The elevating grader is occasionally operated by as few as 12 horses but the result is unsatisfactory, as the normal demand for power when the belt is in full operation exceeds the capacity of 12 ordinary horses, unless the rest periods between loads are long.

EFFECT OF CROSS SECTION OF CUT ON PRODUCTION

Moreover, the ground worked over is seldom level. When this condition prevails, as it does most of the time, even though the material is otherwise such that a full bite might be taken, it is customary to take a full bite only on one side, because the floor of the cut must be reached on a level cross section. The practical result of this is shown in Table 1. Here (omitting all end loads because they are apt to be affected by tapering off) the loading time averaged slightly over 12 seconds on one side as against 15 seconds on the other side—an increase of 25 per cent in loading time on the low side which of course reflects the smaller bite that caused it.

Table 1.—*Typical stop-watch readings on loading time, showing rapid loading, for 16-horse grader with eight 2-horse (1½-yard) wagons*

Average load, 1.35 cubic yards.
Average loading time, 14.9 seconds (343 readings).
Average wagon-exchange time, 16 seconds (296 readings).
Average grader speed, 3.62 feet per second.
Average loading distance, 54 feet.

Loading time, down	Loading time, up	Loading time, down	Loading time, up
Seconds	*Seconds*	*Seconds*	*Seconds*
Turn	Turn	Turn	Turn
28	25	15	15
14	15	11	17
12	15	11	13
13	17	11	14
11	16	14	13
11	20	12	17
12		13	23
12		14	

The manufacturers of elevating graders could reasonably consider a modification of the present design of these machines to enable more rapid lateral adjustment of the plow. With the present designs lateral adjustment is possible but the time required in making the adjustment is too great to permit of its free use. As a result, once the width of the bite is set the shaker can control directly only its depth, which he does by raising or lowering the plow. Consequently, as these machines are commonly operated, if the bite is set 10 inches wide and it is possible to make a cut 14 inches deep on the high side, which will give a cross section of approximately 1 square foot, the cross section on the low side will be reduced in proportion to the amount which must be subtracted from the full depth of the bite in the interest of leveling, since no change is made in the width of the bite. As an illustration, if a cut 3.5 feet on one side and 2.5 feet on the other is to be taken out by standard methods in three levels, the difference between the amount of material secured per foot of run as between the high side and the low side will be about 30 per cent.

If, however, instead of following the custom, the plow is set for a bite wider than normal, the tractor can be driven a little farther from the bank on the high side, thus reducing the width of the bite to normal, and a relatively wide bite may be taken on the low side by merely driving the tractor in normal relation to the bank. The result of following this general practice is to secure a more nearly uniform, as well as a higher, average rate of bite and so a higher average rate of loading. This can be done if the prime mover is a caterpillar tractor, but when horses are used it is difficult of accomplishment.

There is still, of course, the fact that in bringing a cut to grade the layers will taper longitudinally. This further complicates the problem for there must be, on this account, a selection of the length of run for each level and, in general, a tapering out of the bite (see Table 1) at the end of the run. Regardless of other conditions, therefore, it is not possible in highway work either to secure a unform bite or consistently to secure a full bite, but the best shakers give much thought to keeping it as uniform and as large as possible, and the men who are successful in doing this are well worth the relatively high wages they receive.

DEFECTS OF ELEVATING GRADERS

It may be pertinent to mention in this connection that the substitution of tractors for horse-drawn equipment has been hindered by the inadequacies in the design of the elevating grader which have been noted above. The elevating grader was originally designed as a horse-drawn apparatus and as such has functioned quite satisfactorily. It also functions reasonably well with a large caterpillar tractor, for it is a valuable and, on the whole, an effective tool wherever it can be appropriately used. However, the full advantage of the caterpillar tractor as a prime mover will not be secured until the elevating grader is redesigned in some particulars. Basically, the weak points in this machine are the absolute dependence of its elevating mechanism on the bull wheel and the tendency of the belt to foul, which causes it to slip. The performance of the bull wheel depends on the nature of the material over which the machine is operated. It, therefore, follows that no matter how much power is available for hauling the grader and for taking out the large bite necessary if output is to be kept at a maximum, the bite, and therefore the output, is still dependent on the traction which the bull wheel has on the soil over which it is working, and the ability of the machine to transmit this power to the belt. The full advantage of the tractor as a prime mover will not be available to the contractor until the design of the grader is so modified in these particulars that the splendid power plant of the modern caterpillar tractor can be effectively utilized in forcing the grader to pick up and elevate a full load without regard to soil conditions.

The studies made by the bureau's engineers show that under average working conditions a 1½-cubic yard bottom-dump wagon is loaded in about 75 feet (net load measured in place in the cut about 1.3 cubic yards). This indicates that as compared with a cross section of 1 square foot, which the machine can take out under good working conditions, the amount of material placed on the wagons averages a little less than 0.5 cubic foot per foot of run, indicating an effective cutting cross section of slightly less than 0.5 square foot. In other words, the 5,000-cubic yards theoretical daily capacity of the machine is reduced by rather more than 2,500 cubic yards, partly because of general working conditions, in no small degree because of basic inadequacies in the design of the machine itself, and to a considerable degree through inattention to that vitally important detail—the bite. The fact that so large a part of the theoretical capacity of the machine can be lost in this way should serve to emphasize the necessity of giving the closest attention to the bite in order to insure, under all circumstances, the taking of the largest possible bite.

This matter might be of less importance were it not for the fact that, to all intents and purposes, the elevating grader is a constant-speed machine. All of the studies made on horse-drawn equipment indicate that when working under load the speed of operation becomes quite uniform. The same general observation applies to tractor-drawn equipment. The general practice is to establish a working speed and then maintain that speed. Moreover the power output required of the horses is so high that there would be little possibility of increasing the speed to offset a deficiency in the amount of load taken per foot of run, even if other conditions made such a course possible. But, even if ample power were available, there is no possibility of securing any relief in this way because the grader must be operated within the speed at which a wagon can be hauled alongside for loading. Therefore the failre to take out a full bite causes a loss of time and so of output, which can not be compensated by faster moving. Indeed, this loss can not be regained by efficiency in any other part of the work. It is a dead loss.

REDUCTION OF OUTPUT BY SLOW TRAVEL

The next point at which output is lost involves the speed at which the grader is moved. Later on in this article the speed at which the wagons are commonly operated will be discussed in some detail, but it may be observed here that the ordinary walking speed of a team hauling a loaded wagon is from 240 to 250 feet per minute, or about 4 feet per second. The wagon box of an ordinary bottom-dump wagon is a little more than 6 feet long. Loading should begin as soon as the belt is clear of the driver and, while driving at normal speed, the wagon should gain enough on the grader to bring the belt over the back of the wagon by the time the load is finished. This requires the wagon to move about 4 feet farther than the grader during the loading period. This relationship will obtain if the wagon travels about a quarter of a foot per second faster than the elevating grader. The grader should, therefore, maintain a rate of travel of at leat 3¾ feet per second.[3] This rate can be and is maintained by horse-drawn outfits employing 20 horses (see Table 2), but is not generally maintained by the standard 16-horse outfit.

Caterpillar tractors normally maintain a speed of about 3 feet per second, which is about 25 per cent low. This defect is not traceable to any inadequacy in the power plant, but rather to poor governing, which it should be a very simple matter to remedy; but the effect is, nevertheless, to produce a direct loss in excess of 20 per cent in the rate of output, efficiency in other parts of the work being equal, for elevating graders of the design now in common use are not capable of handling any increased bite at this lower speed with which to compensate this loss. Therefore, the contractor who is operating his elevating grader at less than 3¾ feet per second owes his second loss in yardage to this loss, and this, like the loss due to a small bite, is one that can not be compensated by special efficiency in any other part of his work.

TABLE 2.—*Typical grader speeds*

Prime mover	Study No.	Number of study periods [1]	Average grader speed
			Feet per sec.
Caterpillar tractor	13	14	2.9
	17*b*	34	3.3
	18*a*	1	3.1
	18*b*	20	2.8
	20	8	2.9
	23	10	3.0
16-horse team	13	18	3.2
	15	9	3.3
	18	5	3.1
	19	22	3.6
	21	3	3.3
	22	6	2.8
20-horse team	18	5	3.8
	41	5	3.9

[1] The study period was generally 1 hour.

TURNING LOSSES NOT TO BE OVERLOOKED

The third item of loss is loss of time. This occurs at a good many points and in a good many ways. Of these the first is the turn at each end of the grader loop. The time consumed in making a turn may be taken as about 45 seconds and the loss of time due to this cause will run from over 60 per cent of the total time, if the cuts are so short that only one load can be taken on each side, to as little as 6 or 7 per cent as the cuts lengthen to 1,000 feet. If the turn could always be made as a half circle, it could be accomplished in from 30 to 35 seconds. The fact is, however, that the length of productive run on the two sides of the loop is not always the same Where it is not the same the grader has a certain distance to travel on one side of the loop without taking out material. It is a matter of indifference whether the time required for this unproductive run is charged to miscellaneous losses or to turning time. In the bureau's studies it has been charged to turning time because it occurs just before or just following the turns, and on this account the recorded turning time varies a good deal from project to project. In spite of this variation it appears reasonable to consider the turning time, including unproductive run, at 45 seconds. With good superintendence and a fortunate arrangement of material in the cuts, this time may be bettered, but some cases will also be found where the lay of the material to be moved is such that the turn can not consistently be kept within this limit.

While the time consumed is relatively large, the loss due to inefficiency or carelessness in making turns is relatively small on most jobs. On some, however, it does amount to a good deal and can be avoided. As the grader is not under load while turning, the forward gig of a horse-drawn grader can be unhooked and driven around at a trot and the other horses may reasonably be forced to a speed somewhat greater than would be expected when traveling under full load. As between the 16-horse outfit which makes the turn without releasing the forward gig and allows the horses to turn at their pulling pace and the outfit on which the forward gig is removed and the balance of the outfit brought up to a lively walking pace, there is a difference of some seconds in the average turning time.

If caterpillar tractors are used, the turning time is about the same as for horses. This is contrary to the general impression, but results from the fact that the radius of the turn is largely governed by the grader and the ground conditions rather than by the flexibility of the prime mover. In general, the tractor does turn the grader on a somewhat shorter radius than the horses, but as the tractor is ordinarily operated at a slower rate this saving is largely offset. On the other hand, the tractor in high gear can travel quite as fast as the horses even when driven at a rapid walk. It is so easy to shift the tractor into high gear that is is surprising to find that no effort is made to do so on the majority of the jobs. A little time is taken in shifting gears, so the net saving is not large where only the turn is to be accomplished, but if there is in addition any considerable unproductive run, the saving is of more importance. Table 3 shows the turning time on a number of jobs.

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[3] This is based on the assumption that the bite will be maintained at a reasonably high average. If the average bite is small, the difference between wagon speed and grader speed should of course be correspondingly reduced, or, in other words, the speed of the grader should be slightly increased.

TABLE 3.—*Time required to turn elevating graders*

Prime mover	Study No.	Turning time	Remarks
		Seconds	
Caterpillar tractor	13	56	Slow work.
	14	60	Long unproductive runs.
	18*b*	48	Normal.
	20	64	Bad working conditions.
	23	48	Normal.
	30	36	Little unproductive run.
	31*a*	54	An old worn-out machine, very slow.
	31*b*	48	Normal.
16-horse team	13	59	Slow work.
	14	65	Long unproductive run.
	15	45	Normal.
	18	44	Do.
	19	48	Do.
	21	44	Do.
	22	59	Wet ground and slow outfit.
	40	44	Normal.
20-horse team	18	49	Operations a little slow.
	41	43	Normal.

Turning time always is an important factor in output, but it is particularly so when cuts are short. Tables 4 A, 4 B, and 4 C show this quite clearly. If short cuts are common, the percentage of time spent in turning is relatively high and output falls off. If short cuts and short haul are combined with long cuts and long haul, to handle which a contractor carries a large number of wagons, his situation is particularly unfortunate. This matter will be more fully developed in another part of this article, but is here referred to in order to lay stress on the fact that turning time is a factor which the contractor can not afford to overlook. It reduces the output which he can obtain on short cuts and is one of the reasons that light work often fails to yield the expected profit and that projects in localities where elevating graders are uncommon should be carefully scrutinized if it is the intention to use elevating graders in their execution. In such localities it is safe to assume that projects are designed for construction with other types of earth-moving equipment, and contractors accustomed to handling projects that have been designed for elevating graders may easily overlook the fact that the average length of cut and the relative amount of side borrow on a project designed for fresnoes and wheelers is likely to be quite different from the similar items on an elevating grader job.

TABLE 4.—*Effect of length of cut on output and on relation between turning time and working time*

A. AVERAGE MANAGEMENT WITH 1½-CUBIC-YARD WAGONS

[Loading distance, 75 feet; loading time, 23 seconds; exchange time, 18 seconds; time losses, 10 per cent]

N. B.—A loading loop consists of a run down the cut, a turn, a run up the cut, and a second turn. In developing Tables 4 A, B, and C, a half loop—one run and one turn—has been used. By this device the "length of cut" may be compared directly with plans for construction work. The output in loads per hour and the percentages of working time and time used in turning are the same as they would be if the whole loop were used.

Number of loads	Length of cut	Wagon-loading time	Wagon-exchange time	Grader turning time	Time losses due to break-downs, rests, etc., 10 per cent	Total time	Loads per hour	Percentage of total time used in turning grader	Percentage of time grader is at work
	Feet	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Number*	*Per cent*	*Per cent*
1	75	23		45	7	75	48	60.0	30.6
2	150	46	18	45	11	120	60	37.5	38.3
3	225	69	36	45	15	165	65	27.3	41.8
4	300	92	54	45	19	210	69	21.4	43.8
5	375	115	72	45	23	255	71	17.6	45.0
6	450	138	90	45	27	300	72	15.0	46.0
7	525	161	108	45	31	345	73	13.0	46.7
8	600	184	126	45	36	391	74	11.5	47.1
9	675	207	144	45	40	436	74	10.3	47.5
10	750	230	162	45	44	481	75	9.3	47.8
15	1,125	345	252	45	64	706	76	6.4	48.9
20	1,500	460	342	45	85	932	77	4.8	49.4

B. GOOD MANAGEMENT WITH 1½-CUBIC-YARD WAGONS

[Loading distance, 60 feet; loading time, 16 seconds; exchange time, 5 seconds; time losses, 5 per cent]

1	60	16		45	3	64	56	70.3	25.0
2	120	32	5	45	4	86	84	52.4	37.2
3	180	48	10	45	5	108	100	41.7	44.5
4	240	64	15	45	6	130	111	34.6	49.2
5	300	80	20	45	7	152	118	29.6	52.6
6	360	96	25	45	8	174	124	25.8	55.2
7	420	112	30	45	9	196	129	22.9	57.2
8	480	128	35	45	10	218	132	20.6	58.8
9	540	144	40	45	11	240	135	18.7	60.0
10	600	160	45	45	13	263	137	17.1	60.9
15	900	240	70	45	18	373	145	12.1	64.4
20	1,200	320	95	45	23	483	149	9.3	66.4

C. GOOD MANAGEMENT WITH 2-CUBIC-YARD WAGONS

[Loading distance, 90 feet; loading time, 24 seconds; exchange time, 5 seconds; time losses, 5 per ceut]

1	90	24		45	4	73	49	61.6	32.9
2	180	48	5	45	5	103	70	43.7	46.6
3	270	72	10	45	6	133	81	33.8	54.1
4	360	96	15	45	8	164	88	27.4	58.5
5	450	120	20	45	9	194	93	23.2	61.9
6	540	144	25	45	11	225	96	20.0	64.1
7	630	168	30	45	12	255	99	17.6	65.9
8	720	192	35	45	14	286	101	15.7	67.2
9	810	216	40	45	15	316	103	14.2	68.4
10	900	240	45	45	17	347	104	13.0	69.2
15	1,350	360	70	45	24	499	108	9.0	72.2
20	1,800	480	95	45	31	651	111	6.9	73.6

Another loss of time comes in loading the wagons. The time required to load a wagon depends on the cross section of the bite, the speed at which the grader is drawn, and the percentage of material that reaches and remains on the wagons. The first two of these items have been noted and discussed above. The third, the percentage of material which is delivered to and retained on the wagons, while perhaps of minor importance, nevertheless is of sufficient significance to warrant attention. Material is lost in important quantities back of the disk, along the edge of the belt, and by spilling from the wagon. The first of these losses is sufficient to justify manufacturers in considering a more effective design, particularly around the lower end of the belt. Contractors often avoid some of the loss of material which takes place at this point by placing a steel plate over the rather wide opening back of the disk. This simple expedient serves to save a good many loads of material during the course of a day's work, particularly if the machine is working in light soil. No convenient method of measuring these losses has been devised, but from such observations as have been made it appears that they may amount to from 5 to 10 per cent of the material taken out by the disk, particularly in light loam and in sandy soils.

When the bite is such that wagons are loaded in 75 feet, a grader speed of 3 feet per second results in a loading time of 25 seconds. If the grader speed is 3¾ feet per second, the loading time is 20 seconds. The average loading time as determined by the studies is about 23 seconds. If the bite is increased so that a load can be had in 60 feet, and the grader speed is maintained at 3¾ feet per second, the loading time drops to 16 seconds. As a production of 600 to 700 loads per day is quite common, the cumulative effect of slow loading is considerable.

PROMPT EXCHANGE OF WAGONS ESSENTIAL

Another important factor in any summary of time losses is the period during which wagons are exchanged at the grader. This exchange period averages about 18 seconds (see Table 5). It includes the most conspicuous and, perhaps, the most easily avoidable losses which the contractor encounters in elevating grader work. Theoretically, it is possible to exchange wagons in about 4 seconds, if the replacement wagon follows the loading wagon with the noses of the horses hard against the tailboard. From the horses' heads to the back of the driver's seat is about 14 feet and, allowing 3 feet for the width of the belt, the replacement team has only to move about 17 feet to spot the next wagon which, at ordinary walking speed, requires a little over 4 seconds. As a matter of observed practice (see Table 6C), the wagon exchange may be made within this time. As compared with this attainable exchange period, however, it is found that the exchange time is generally not less than 11 or 12 seconds over any extended period, even though the wagon supply is adequate, and often when the wagon supply is inadequate the time required is much greater, as may be seen from Table 5.

Table 5.—*Effect of wagon supply on exchange time*

Study number	General average exchange time	Average exchange time when wagon supply was adequate	Study number	General average exchange time	Average exchange time when wagon supply was adequate
	Seconds	*Seconds*		*Seconds*	*Seconds*
13	17.2		18d	21.4	
13a		11.6	19	16.0	
13b		13.3	20	17.9	10.1
14		10.8	21	38.1	12.8
14a	18.0		22	51.8	11.6
15	15.8	14.0	23	10.2	7.5
17a	19.2		30		7.8
17b		7.1	31a	22.0	
18a	8.9		31b	18.2	
18b	34.7	11.5	40	13.5	
18c	21.0	9.9	41	9.0	

The common failure to exchange wagons within a reasonable period, say within an average of 5 seconds, is due to a number of causes. Of these, the first is a positive inadequacy in the wagon supply. For the effect of this see Table 6A. Conversations with numerous contractors leave the impression that there is a surprising tendency to view the elevating grader as a piece of equipment which can be used to load a train of wagons—that is, that the wagon train is the basis on which calculations are made. The correct view, of course, is that the elevating grader is the primary tool and that the wagon supply should be selected to handle all the material the grader can be made to produce. If the wagon supply is inadequate it is, of course, impossible to keep the exchange time down because the time spent in making an exchange is governed by the rate at which the wagons can be supplied to the grader.

An equally important factor is the superintendence. The lack of good superintendence is quite as conspicuous on elevating grader work as in the operation of other types of earth-moving equipment. On the jobs studied it has been the rule rather than the exception to find replacement teams walking from 20 to 100 feet behind the wagons which they are to replace. If a team is 40 feet behind its leading wagon, 10 seconds are required at the customary walking rate of 4 feet per second to reach the point from which the exchange should have begun; that is, the tailboard of the loading wagon. These 10 seconds must be added to the 4 seconds which are required for the move forward from the tailboard of the loading wagon to the loading position. The larger part of the difference between the proper exchange time and the actual exchange time when the wagon supply is adequate is to be accounted for in just this way.

Table 6.—*Stop-watch readings of wagon exchange time*

A. SHOWING HIGH EXCHANGE TIME OCCASIONED BY INADEQUATE WAGON SUPPLY WITH CONSEQUENT LOW PRODUCTION

Wagon exchange time cutting toward dump	Wagon exchange time cutting away from dump	Wagon exchange time cutting toward dump	Wagon exchange time cutting away from dump
Seconds	*Seconds*	*Seconds*	*Seconds*
15	13	22	17
15	11	15	14
8 min. 50 sec.[1]	8	12	15
[1] 32	15	12	7 min. 50 sec.[1]
10	18	15	10
15	20	25	
1 min. 0 sec.[1]	12		

[1] Grader waiting for wagons.

Average wagon exchange time, 56 seconds; loading distance, 55 feet; number of loads per hour, 51; length of cut, 275 feet; wagon-haul distance, 1,600 feet.

B. SHOWING UNNECESSARY DELAY IN EXCHANGE TIME WHEN CUTTING AWAY FROM DUMP OCCASIONED BY IMPROPER TURNING OUT OF LOADED WAGON

Wagon exchange time cutting toward dump	Wagon exchange time cutting away from dump	Wagon exchange time cutting toward dump	Wagon exchange time cutting away from dump
Seconds	*Seconds*	*Seconds*	*Seconds*
7	25	10	23
8	26	6	23
8	25	7	32
7	27		
8	30	Average 7.4	Average 26.0
6	23		

Length of cut, 660 feet; wagon-haul distance, 1,000 feet.

C. SHOWING AVERAGE EXCHANGE TIME OBTAINABLE WITH GOOD MANAGEMENT

Wagon exchange time cutting toward dump	Wagon exchange time cutting away from dump	Wagon exchange time cutting toward dump	Wagon exchange time cutting away from dump
Seconds	*Seconds*	*Seconds*	*Seconds*
6	6	4	5
4	5	4	7
4	4	6	5
5	4	5	4
5	5		
6	4	Average 4.9	Average 4.9

That part not so accounted for is attributable to one of two causes. The first is the common failure to insist that the load shall be finished with the belt over the end of the wagon. If it is finished with the belt just back of the driver, it is hard to make the exchange in less than 6 seconds. The second is the manner in which the loaded wagon is turned out when the grader is moving away from the dump. If while working in close quarters this turn is made before the replacement wagon is in loading position, the exchange takes about 25 seconds. (See Table 6B.) If however, the loaded wagon is driven ahead at an angle until the replacement wagon can reach the loading position and then stands until the loading wagon and its replacement pass, the loss of time affects only the loaded wagon. The grader waits for the next wagon no longer than on the trip toward the dump. Figure 2 shows the correct turn and two incorrect turns.

Proper position of replacement wagon for prompt wagon exchange

Improper position of replacement wagon for prompt wagon exchange

It should be observed in this connection that if a contractor has supplied a wagon train with an elevating grader under the presumption that his exchange period will approximate 18 seconds and then, by careful supervision, endeavors to reduce his exchange time to a correct average—5 seconds or under—he will find that he is more or less handicapped in doing this because saving nearly a quarter of a minute per load by the reduction of his exchange time will at once create a deficiency in the wagon supply. This is mentioned because it has been observed that contractors who have endeavored to reduce the exchange time have, in some instances, more or less abandoned the effort as impossible of attainment because of this fact. There is little use in trying to put snap into the wagon exchange unless there are enough wagons on the job so that savings made through efficient exchange are not at once obliterated by inadequacy in the supply.

RELATION OF WAGON SUPPLY TO LENGTH OF HAUL A CONTROLLING FACTOR IN EXCHANGE TIME

Securing an exchange period of 5 seconds will not prove a simple matter. A brief study of Figure 1 will show the reason for this. The loading loop varies in length from one so short that only one load can be taken on each side, to a loop 1,000 feet or more in length. The wagon loop also varies in length in the same way. Take, as an illustration, a loading loop 500 feet in length from which the haul is in one direction to a point 500 feet below the lower end of the loading loop. Under these conditions the wagon haul will vary from 500 to 1,000 feet in length and will average something over 750 feet. If, therefore, the wagon supply is calculated on the basis of the actual average haul, it will be excessive at some points and deficient at others. In order to obviate this difficulty the contractor should require empty wagons to trot to position whenever necessary to place a wagon in position behind the loading wagon before the load is completed. If the wagon supply is adequate for the actual length of haul involved, this will enable an efficient superintendent to avoid loss of time in the exchange of wagons.

If the wagon supply is adequate for the average haul prevailing, there need be no fear that the stock will be injured by this practice. However, if trotting is habitually practiced in order to increase production in the face of a deficient wagon supply, the effect on the stock is likely to be disastrous, and such a practice is not to be recommended. There is a difference between this and the practice of trotting the teams for the purpose of adjusting an adequate average wagon supply to the fluctuating length of wagon haul that is a necessary feature of elevating grader work. A horse can turn out a large amount of power or work at a rapid rate for a short time if the rest periods are such that the day's output of power is not excessive. Trotting, in order to adjust the wagon supply to the changing haul distance, is within his normal capacity and may reasonably be practiced, for the extra speed required on one trip will be offset by a corresponding period of rest on the next. But if the wagon supply is inadequate, there is no corresponding rest period, for the time gained in trotting results in a positive increase in the power output required during each working period. If the power output resulting from this practice is uniformly excessive over any extended period, the stock will be correspondingly damaged.

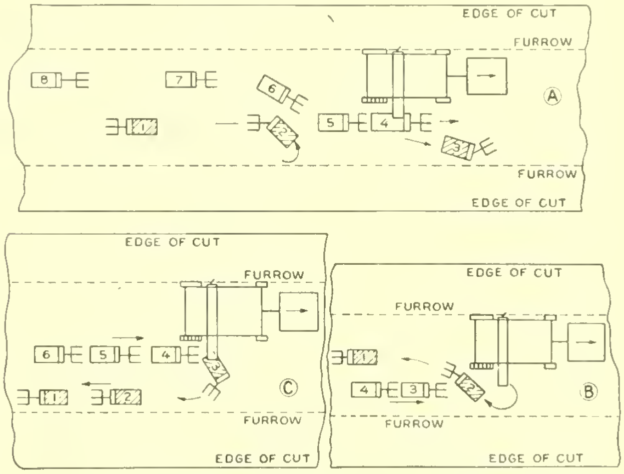

FIG. 2.—Methods of turning loaded wagons away from grader: A. The correct method. No time lost by grader. B. Incorrect method in a narrow cut. Time lost by grader about 20 seconds. C. Incorrect method in a wide cut. Time lost by grader about 10 seconds.

MISCELLANEOUS DELAYS

Finally, delays of one sort or another account for time losses of considerable importance. These fall principally under three headings—breakdowns, cleanouts, and rests.

Breakdowns occur rather frequently on elevating grader work. The chain drives give the most trouble. The belt gives more or less. Other parts break occasionally. Lack of proper maintenance is the outstanding cause of this trouble. Instead of buying a new set of chains for the belt drive every spring, the old set will be continued in service until it is replaced link by link as it breaks on the job. It takes only two or three minutes to repair such a break but, in the meantime, a dollar's worth of output has been lost. On one job the writer found the grader standing idle and the crew in camp. The rear axle had broken—not an uncommon occurrence—and the job had been delayed the better part of a week because no extra axle was on hand. On another job the grader was idle because the main drive pinion had worn out. The contractor had known of its condition for weeks, but hoped it would last a little longer. The repairs cost both the contractor and his men a good deal more than they would had they been made at the proper time. To pay a dollar a link, in lost time, for repairing drive chains, to lose a week's time because repair parts are not on hand, or to hope that a worn-out pinion will somehow last a few weeks longer are all illustrations of the general tendency to under-maintain the elevating grader, a tendency that expresses itself in constantly recurring breakdowns with their attending loss of time.

Elevating graders are often continued in service long after they have become so badly worn that breakage is frequent and trouble with the elevating mechanism more or less chronic. The result is not only the definite time losses noted above, but a constant, rather intangible loss of efficiency rather hard to prove definitely, which expresses itself in a number of ways. Thus, contractors often object to the operation of tractors at speeds as high as 3¾ feet per second on the ground that when obstructions are encountered the grader is likely to be broken. Of course, if the grader itself is in good condition, this is a matter which can readily be obviated by using a proper spring coupling between the grader and the tractor and, on new machines, by an appropriate redesign of the devices by which the plow itself is held in place. These objections reflect the fact that many of the graders now in use are light and more or less worn out, and that there is a valid reason to fear the result of placing the machine under heavy strain.

This same general observation applies to the objection, often raised, to taking a full bite in heavy ground when other conditions would permit; that is, that the grader is not strong enough to stand the heavy strain thus imposed on it. For many of the machines now in use, this is no doubt the case. But, as the objective is yardage at the lowest possible cost, it is poor economy to continue an old machine in service if this can be done only by accepting a reduction in output. At current prices, a substantially built machine can be had for the money secured from the movement of from 8,000 to 10,000 cubic yards of material. The loss of as little as 50 or 60 yards a day during the construction season will cover the entire cost of a substantial machine—a very small amount when considered in the light of the heavy reduction in rate of output which attends any reduction in the bite.

It is not necessary to multiply illustrations in order to emphasize the point that the tendency to reduce production in order to get along with a worn-out machine often generates intangible losses of even greater importance than the positive time losses due to breakdowns. The remedy is obvious. Obsolete and worn-out graders should be replaced and a good grader should receive the attention needed to keep it always at a high point of efficiency.

The clean out around the lower roller stops the outfit a good many times every day. In moist clay soils it is sometimes necessary to clean the whole bottom side of the belt. These operations are caused by the tendency of droppings to accumulate just above the bottom roller and on the belt itself. These accumulations, which cause the belt to function improperly, must be removed from time to time—occasionally as often as once for every 10 or 15 wagons loaded. The clean out takes from a minute to three or four minutes. In the course of a day's work the loss of time from this cause may therefore be considerable. It is not, however, an avoidable loss with the present design of the grader.

RESTING STOCK

Another delay, involving heavy losses, results from the resting of stock. This is most conspicuous when the grader is drawn by 16 horses. Not much will be said on this point, as the dynamometer studies now in progress should yield precise information on the power output required. It may be observed, however, that, whereas the time losses on caterpillar-tractor jobs tend to run in the neighborhood of 5 per cent and are caused almost entirely by grader breakdowns and clean outs, the time losses on horse-drawn jobs sometimes exceed 20 per cent in hot weather. As clean-out losses are independent of the motive power and breakdowns certainly no more frequent on the horse-drawn jobs, the importance of these rest periods from the standpoint of output will be apparent.

The horse as a power unit has certain limitations, it appears, which tend to interfere with high production. When 16 horses are attached to an elevating grader, it is possible for them to develop the power necessary in order to load wagons promptly if the rest periods between loads are long enough. But when the weather is hot or the flow of power becomes more constant, i. e., as increasing efficiency of superintendence tends to reduce the wagon-exchange period, the burden placed on a 16-horse outfit tends to become excessive and the extra rest periods must be taken. These, of course, tend to offset any advantage secured from reduced wagon-exchange time. For this reason if an effort is to be made to develop high efficiency additional horse power ordinarily must be supplied. This is particularly true during the summer months. Observations of 20-horse outfits tend to establish the fact that an outfit of this size can endure the increased strain incident to high output. But even when 20 horses are used rest periods may have to be permitted during hot weather. It is here that the caterpillar tractor shows its great advantage. It does not tire as the percentage of time under full load is increased and hot weather has no adverse effect on it. Where a high degree of efficiency is to be maintained and with it a high rate of output, the caterpillar tractor offers a source of power at once dependable and effective. Its slow rate of travel, while a disadvantage, by no means outweighs this advantage, and, in any event, there is no reason to suppose that its speed is necessarily slower than that of well-driven horses.

In summarizing the question of time losses due to breakdowns, clean outs, and resting stock, it may be noted that these losses vary over rather wide limits due to soil conditions, the age of the machines worked, weather, etc. In general, 5 per cent may be appropriately allowed on caterpillar-tractor jobs of which, normally, less than half will be clean out losses. If 16 horses are to be used on the grader, another 5 per cent should be added for resting stock on a full-season job where soil conditions are good. If the job is short and is to be done during hot weather, 10 per cent should be allowed in normal loam and clay, but if the ground is heavy, even 15 per cent may be inadequate.

PRACTICES THAT EAT INTO THE WORKING DAY

The losses discussed above affect production during the working period. In addition to these losses the contractor not infrequently accepts practices which effectively reduce the working period itself. This may be illustrated by citing the practices on a job where the nominal working period was 10 hours. The men understood that 10 hours work was required and fractional days were paid for on that basis. The contractor made it a practice to move camp about every four miles, working some two miles in each direction from the camp, which resulted in an average trip to and from camp of about a mile. This trip at the ordinary walking speed for the horses took about 20 minutes. It was the custom to make the trips out—morning and noon—on the contractor's time and the return trips on the men's time. Here then, was a direct loss of 40 minutes of working time every day. Also, it was customary to water the stock morning and afternoon. This stopped operations for from 15 to 25 minutes twice a day—a net loss of perhaps 40 minutes a day. Watering stock during the working period is not a general practice among contractors, even during hot weather, but the fact that it is sometimes done with no modification of the nominal daily working period suggests that where managerial practices of this sort prevail the working period should not be considered 10 hours. In this case it really averaged about 8 hours and 40 minutes.

It has been pointed out that, on work of this sort, losses are not recoverable. If the bite is small there is a loss of time and output because too much time is used in loading the wagons. If the speed of the grader is too low there is the same effect. If time is lost in wagon waits or in delays, output is reduced because fewer loads can be taken out. In all of these cases the losses are positive and cumulative, for the men employed must be paid as much and the horses will eat as much when the production is low as when it is high. This also applies to lax managerial policies. The contractor who is paying for 10 hours of work and getting only 8⅔ hours is suffering a direct loss which will appear as a corresponding reduction in output.

THE SITUATION SUMMARIZED

Summarizing the situation briefly, there are comparatively few contractors now operating elevating graders who succeed consistently in obtaining an average output much over 60 loads an hour. Many contractors never consistently reach this output. But, as compared with this rate of output, there is no apparent reason why, by careful attention to the speed at which the grader is moved and the bite that it takes, a load can not be consistently put onto a wagon in 16 seconds and an exchange of wagons made in 5 seconds which, with due allowance for turns, and the elimination of unnecessary delays, should enable an efficient contractor to produce a load every 30 seconds or 120 loads an hour. In order to secure such a rate of production the principal requirements are an adequate wagon supply and really competent superintendence. In Table 7 the relationships which have been discussed are set down so as to show the effect of these various elements on output. An examination of this table will show that if a large output is to be secured it must be done by taking a consistently heavier bite, by maintaining a proper grader speed, by making a prompt wagon exchange and by reducing the time losses to a minimum.

TABLE 7.—*Effect of various losses on elevating grader output*

	Grader speed 3¾ feet per second			Grader speed 3 feet per second		
	Bite 40 per cent	Bite 50 per cent	Bite 60 per cent [1]	Bite 40 per cent	Bite 50 per cent [2]	Bite 60 per cent
Theoretical maximum production of grader at plow, cubic yards per 10-hour day	5,000	5,000	5,000	4,000	4,000	4,000
Rate at which material is delivered by grader while it is loading wagons, cubic yards per 10-hour day	2,000	2,500	3,000	1,600	2,000	2,400
For 1½ cubic yard wagons:						
Portion of grader time spent in loading wagons (based on average length of cut of 450 feet) per cent	--------	--------	--------	50	45	42
Portion of grader time that can, under good management, be spent in loading wagons (450-foot cut) per cent	62	60	58	--------	--------	--------
Resulting average production per 10-hour day as now obtained cubic yards	--------	--------	--------	800	900	1,010
Average production per 10-hour day reasonably possible with good management cubic yards	1,240	1,500	1,740	--------	--------	--------
For 2 cubic yard wagons:						
Portion of grader time spent in loading wagons (based on average length of cut of 450 feet) per cent	--------	--------	--------	57	53	50
Portion of grader time that can, under good management, be spent in loading wagons, (450-foot cut) per cent	64	63	62	--------	--------	--------
Resulting average, production per 10-hour day cubic yards	1,280	1,575	1,860	910	1,060	1,200

[1] Contractors should endeavor to work in this field.

[2] Contractors now customarily work in this general field.

The relations shown in Table 7 are based on a 450-foot loading loop. During the past working season studies, some short and some covering a period of a month or more, were made on more than 20 separate outfits. The average length of cut during these studies (the loading loop) was almost exactly 450 feet. But, as between the various studies, the length of cut averaged under 300 feet on 2, between 300 and 400 feet on 5, between 400 and 500 feet on 5, between 500 and 600 feet on 3, and over 600 feet on 5.

Tables 4A, 4B, and 4C give additional data on the production that can be secured with average superintendence and high-class superintendence for various lengths of cut. As it is wholly impossible to secure as great an output on short cuts as on long ones, it seems

(Continued on page 41)

PROCEDURE FOR TESTING SUBGRADE SOILS

A DESCRIPTION OF THE REVISED METHODS ADOPTED BY THE BUREAU OF PUBLIC ROADS

By J. R. BOYD, Assistant Engineer of Tests U. S. Bureau of Public Roads

METHODS employed by the Bureau of Public Roads in testing subgrade soils were first described in a paper by A. T. Goldbeck and F. H. Jackson in the Proceedings of the American Society for Testing Materials, volume 21, 1921. A more complete description was given by the writer in a paper published in the Proceedings of the American Society for Testing Materials, volume 22, 1922, entitled "Physical Properties of Subgrade Materials." Since the publication of this description a number of changes have been found desirable, and these changes are incorporated in the following description of the revised methods.

PREPARATION OF SOIL SAMPLES

1. The sample as recieved from the field is dried in an oven at a temperature not exceeding 100° C. (212°F.) It is then broken up in a mortar by means of a pestle (preferably rubber-covered), care being taken not to break any of the rock fragments, and passed through a ¼-inch screen. The fractions passing and retained on the screen are then weighed separately and the weight of the latter, expressed as a percentage of the weight of the entire sample, is recorded for use in the mechanical analysis. If it is desired to keep the material retained by the ¼-inch screen, it should be kept separate from the remainder of the sample.

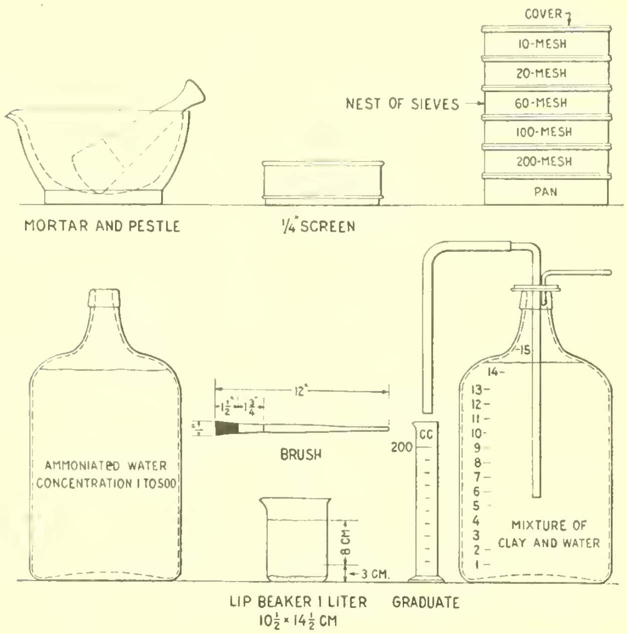

FIG. 1.—Apparatus for determining the mechanical analysis of subgrade soils

2. From the material passing the ¼-inch screen, a sample is selected, by the method of quartering, sufficiently large to make as many of the subgrade tests as may be desired. This sample is passed through the 10-mesh sieve, and the material retained on this sieve is discarded. With the exception of the mechanical analysis all subgrade tests are made on material which passes the 10-mesh sieve.

MECHANICAL ANALYSIS

In making a mechanical analysis (the apparatus employed is shown in Fig. 1) of subgrade soils, four fractions are determined; namely, coarse material, sand, silt, and clay. These terms are defined as follows:

> Coarse material is all material retained above the 10-mesh sieve.
>
> Sand is that material which passes a 10-mesh sieve, is retained on the 200-mesh sieve, and subsides through a height of 8 centimeters in ammoniated water (concentration 1:500) in 8 minutes.
>
> Silt is that material, which passes a 200-mesh sieve, and subsides through a height of 8 centimeters in ammoniated water (concentration 1:500 in 8 minutes.
>
> Clay is that material which passes a 200-mesh sieve and which does not subside through a height of 8 centimeters in ammoniated water (concentration 1:500) in 8 minutes.

It has been found by experience that a more accurate mechanical analysis is obtained if the analysis is made on material passing the ¼-inch screen instead of material passing the 10-mesh sieve. The reason for this is that without the action of water it is impossible to separate completely the fine material from that retained on the 10-mesh sieve. In view of this fact the mechanical analysis is made in the following manner:

From that part of the entire sample which has passed the ¼-inch screen, as described in paragraph 1, "Preparation of Soil Samples," a sample is selected, by the method of quartering, weighing approximately 50 grams.

This sample is placed in an oven and dried to constant weight at a temperature not exceeding 100° C. (212° F.). After cooling to room temperature in a desiccator it is weighed and placed in a beaker with approximately 500 cubic centimeters of distilled water. (Where clear tap water is available, distilled water is not necessary.) This mixture is gradually brought up to the boiling point during a period of one hour and allowed to simmer for an additional hour, but actual boiling should be avoided. After standing until cool the supernatant liquid is decanted to a depth of 3 centimeters above the bottom of the beaker into a large vessel (10 to 20 liters). Ammoniated water (concentration 1:500) is then added to the beaker to a height of 11 centimeters and the material is thoroughly brushed and dispersed with a stiff brush for a period of 1 or 2 minutes and then allowed to stand for 8 minutes. The supernatant liquid is then decanted to a depth of 8 centimeters from the surface of the liquid into the large vessel. Ammoniated water is again added to the beaker, brushing repeated as before, and after 8 minutes sedimentation, the supernatant liquid is decanted again into the large vessel. This process is repeated until the supernatant liquid becomes clear after 8 minutes sedimentation. The "sand" and "silt" have now been separated from the "clay" which is contained in the large vessel. The material in the beaker (sand and silt) is transferred to an evaporating dish and dried to constant weight at a temperature not exceeding 100° C. (212° F.), cooled in a desiccator and a sieve analysis made, using the 10-mesh, 20-mesh, 60-mesh, 100-mesh, and 200-mesh sieves. The time of shaking for the sieve analysis is 15 minutes.

The material retained on the 10-mesh sieve is coarse material and should be expressed as a percentage of the entire sample in order that it may be added to the percentage of material retained on the ¼-inch screen, thus giving the total percentage of coarse material existing in the original sample. This is accomplished by subtracting the percentage of material retained on the ¼-inch screen from 100 and multiplying the percentage of material retained on the 10-mesh sieve, as determined from the small sample, by the factor obtained from this subtraction.

The percentage of "sand," "silt," and "clay" should be computed on the basis of the weight of the fine material only, which means that the weight of the material retained on the 10-mesh sieve should be subtracted from the weight of the small sample before computing the percentages of the fine material.

The large vessel is thoroughly shaken until all material is in suspension. Two hundred cubic centimeters of the liquid are then siphoned from approximately the center of the liquid into a graduate. The liquid is transferred from the graduate to an evaporating dish, evaporated to dryness, and the weight of the clay residue determined. This weight multiplied by the factor obtained by dividing the total volume of liquid by 200 represents the total weight of clay in the sample, and this weight should be expressed as a percentage of the fine material by dividing it by the weight of the material passing the 10-mesh sieve.

The sum of the percentages of sand and silt subtracted from 100 should equal the percentage of clay as actually determined. A maximum error of 3 per cent is allowable In order that the analysis may total 100 per cent, the error should be distributed logically among the fractions.

The actual determination of clay may be eliminated, and the percentage may be computed from the following formula:

$$\frac{\text{Original weight of fine material} - (\text{weight of sand} + \text{weight of silt})}{\text{original weight of fine material}} \times 100$$

but this method should be used only as a last resort since it eliminates the check.

MOISTURE EQUIVALENT TEST

The moisture equivalent of a soil is defined as the amount of moisture, expressed as a percentage of the dry weight of the sample, retained by a soil when it is subjected to a centrifugal force equal to 1,000 times the force of gravity. The apparatus used in the test is shown in Figure 2, and the procedure for determining the moisture equivalent is as follows:[1]

A 5-gram sample of the soil, prepared as described under "Preparation of soil samples," paragraph 2, is placed in a Gooch crucible, the bottom of which has been previously covered with a piece of filter paper, and allowed to take up moisture until completely saturated. It is then placed in a damp closet over night to insure a uniform distribution of moisture throughout the soil mass, after which the crucible is placed in a Babcock cup, in the bottom of which is a rubber stopper, shown in Figure 2, with a hole through its center sufficiently large to hold the water thrown out by the centrifuge. Besides receiving the ejected water, the stopper also serves as a cushion. The Babcock cup is provided with a brass cap to prevent evaporation. The sample is centrifuged for a period of one hour, at a speed which, for the diameter of head used, will exert a centrifugal force 1,000 times the force of gravity, upon the center of gravity of the soil sample. Immediately after centrifuging, the sample is weighed, dried in an oven to constant weight at a temperature not exceeding 100° C. (212° F.), and a second weight determined. The moisture equivalent of the soil is calculated from the following formula:

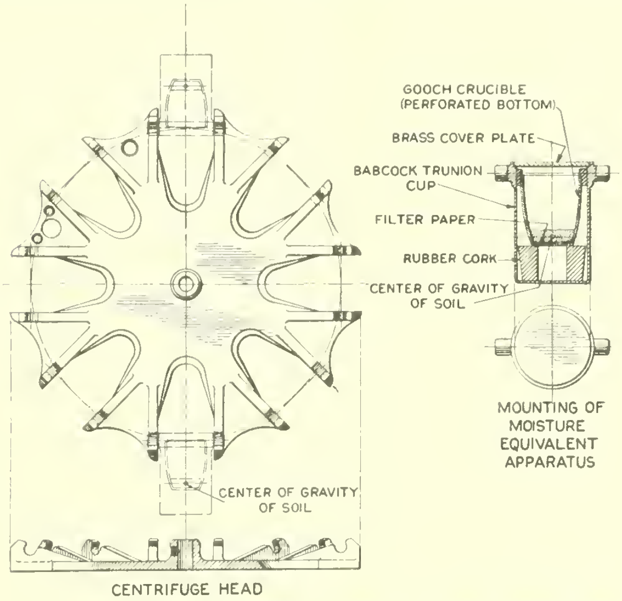
GOOCH CRUCIBLE (PERFORATED BOTTOM)
BRASS COVER PLATE
BABCOCK TRUNION CUP
FILTER PAPER
RUBBER CORK
CENTER OF GRAVITY OF SOIL
MOUNTING OF MOISTURE EQUIVALENT APPARATUS
CENTER OF GRAVITY OF SOIL
CENTRIFUGE HEAD

Fig. 2.—Apparatus for determining the moisture equivalent of subgrade soils

$$\text{Moisture equivalent} = \frac{(A-b)-(A^1-b^1)}{A^1-(a+b^1)} \times 100$$

in which

A = weight of crucible and contents after centrifuging.
A^1 = weight of crucible and contents after drying.
a = weight of crucible.
b = weight of filter paper wet.
b^1 = weight of filter paper dry.

The test should always be made upon duplicate samples and these should be placed opposite to each other in the centrifuge, care being taken to accurately counterbalance them.

The variation between the two values obtained should not exceed 1 per cent for values of the moisture equivalent up to 15 per cent, and 2 per cent for values above 15 per cent.

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[1] A practical field test for determining the approximate moisture equivalent percentage of soils is described in Public Roads, vol. 5, No. 6, August, 1924, p. 10.

CAPILLARY MOISTURE TEST

The capillary moisture of a soil may be considered as the amount of moisture the soil is capable of taking up against the force of gravity. All subgrade soils have the ability to take up capillary moisture to some extent, and it is for the purpose of determining the relative amounts which can be taken up by subgrade soils of varying characteristics that the following test is used. The apparatus is shown in Figure 3.

A sample of the soil, prepared as described under "Preparation of soil samples," paragraph 2, is dried to constant weight in an oven at a temperature not exceeding 100° C. (212° F.), allowed to cool in a desiccator, and then poured loosely into a glass tube (inside diameter 25 millimeters), the bottom end of which has previously been covered with cheesecloth. The tube is jarred until the soil reaches a height of 10 centimeters and no further settlement takes place. The lower or covered end of the tube is then just immersed in water. When capillary moisture reaches the top of the soil column, the tube is weighed daily until it comes to constant weight. (A slight fluctuation in weight is to be expected due to barometric changes.) The percentage of capillary moisture is then computed from the following formula:

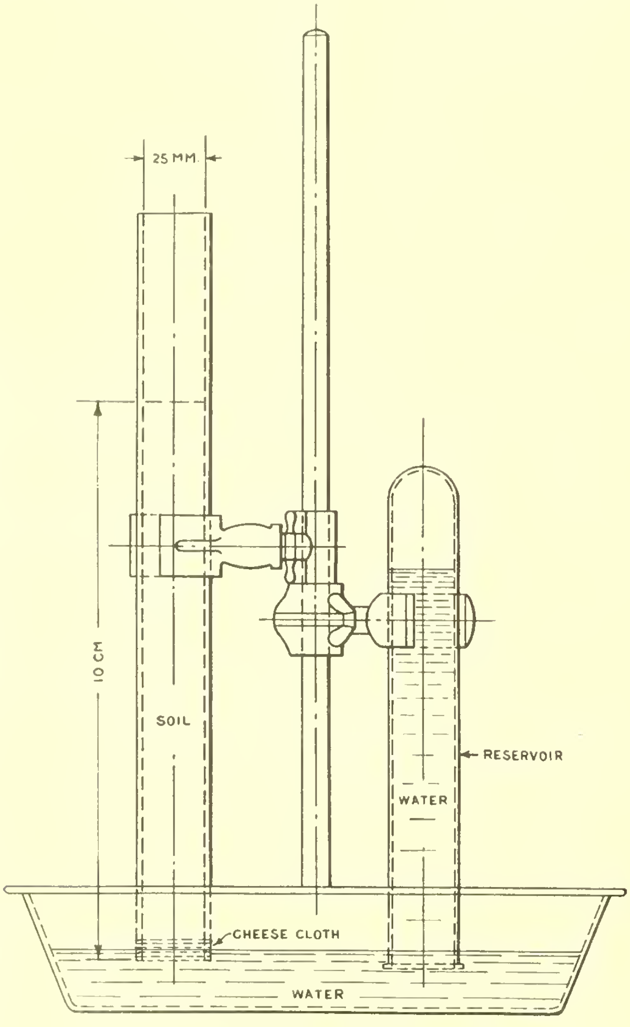

FIG. 3.—Apparatus for the capillary moisture test

$$\text{Percentage of capillary moisture} = \frac{(A-A^1)-(b-b^1)}{A^1-(a+b^1)} \times 100$$

in which

A = (weight of glass tube) + (wet soil) + (cheesecloth and rubber band wet).
A^1 = (weight of glass tube) + (dry soil) + (cheesecloth and rubber band dry).
a = weight of glass tube.
b = weight of cheesecloth and rubber band wet.
b^1 = weight of cheesecloth and rubber band dry.

VOLUMETRIC SHRINKAGE TEST

A very important characteristic of subgrade soils is their relative shrinkage when moisture is withdrawn from them. The test for determining this characteristic is made in the following manner:

A sample of soil, prepared as described under "Preparation of soil samples," paragraph 2, is thoroughly mixed with a given quantity of water and placed in a small, flat, cylindrical, porcelain dish approximately 6 centimeters in diameter and 1 centimeter deep and struck off with a spatula. (The porcelain dish should be greased with a film of vaseline, or similar material, previous to filling it with soil.) The sample is then allowed to dry in the air and is weighed at frequent intervals until no further loss of weight is recorded. If desirable the sample may be placed in the oven after air drying for 24 hours.

The percentage of volumetric contraction of the soil is determined as follows: The weight of mercury required to just fill the porcelain dish is obtained, as well as the weight of mercury which is displaced by the soil pat after shrinkage takes place.

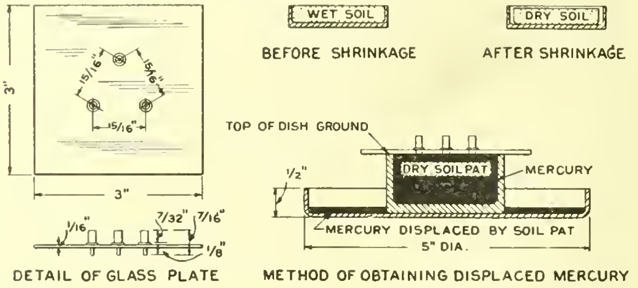

FIG. 4.—Apparatus for determining the volumetric shrinkage of subgrade soils

The volume of mercury required to just fill the porcelain dish and also the volume displaced by the soil pat is determined by means of the apparatus shown in Figure 4. The weights of these two volumes are also determined, denominated A and B, respectively, and the volumetric shrinkage of the soil is computed from the following formula:

$$\text{Percentage of volumetric shrinkage} = \frac{A-B}{A} \times 100$$

Two samples are always prepared for each soil and the average result considered as the percentage of volumetric shrinkage of the soil.

In some cases it may be desirable to know the percentage of lineal shrinkage; consequently, a curve has been prepared, Figure 5, showing the relation between the percentage of volumetric shrinkage and the percentage of lineal shrinkage.[2]

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[2] A practical field method for determining the lineal shrinkage of soil is described in Public Roads, vol. 5, No. 6, August, 1924, p. 12.

COMPARATIVE BEARING VALUE TEST

This test is used for determining the *comparative* bearing power of soils and should not be considered as indicative of the actual field bearing power. It is made in the following manner:

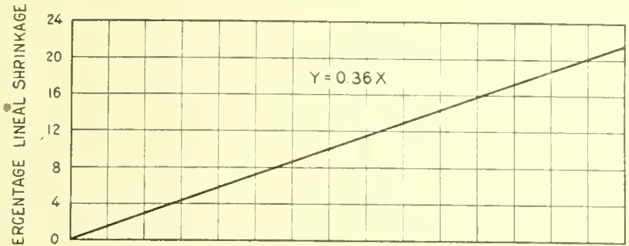

Fig. 5.—Relation between percentage of volumetric shrinkage and percentage of lineal shrinkage of subgrade soil

The soil, prepared as described under "Preparation of soil samples," paragraph 2, is mixed by hand with the desired amount of water. The mixture is thoroughly kneaded for several minutes in order that complete distribution of the moisture may be obtained. It is then covered with a pan and allowed to stand for 15 minutes, after which it is placed in a cylindrical brass container and subjected to an initial load of 10 pounds per square inch to insure uniformity of compaction. (Fig. 6.) After putting the 1-square-inch bearing block in position the container is mounted in the bearing value apparatus (Fig. 7) so that the point of application of the load on the horizontal beam is 0.1 inch above the position of equilibrium. The sand for applying the load is released and simultaneous readings of load and penetration are made for each one-tenth-pound increment of load, the maximum penetration used being 0.2 inch. As the point of equilibrium of the system corresponds to a dial reading of zero, the beam is displaced 0.1 inch above the true horizontal position at the beginning of the test and 0.1 inch below at the end of the test. Load readings as obtained from the spring balance must be multiplied by 4, due to the ratio of the lever arms. Check tests are run on each soil, and from the average of these the bearing value curve is plotted. For comparative purposes the bearing value of the soil is taken as the load in pounds per square inch required to produce a penetration of 0.1 inch.

SLAKING VALUE TEST

The rapidity with which a soil slakes down under the action of moisture is considered as one of its important characteristics and is determined in the following manner:

A sample of the soil prepared as described under "Preparation of Soil Samples," paragraph 2, is thoroughly mixed with its moisture equivalent moisture and molded, under a pressure of 1,875 pounds per square inch, into a briquette $1\frac{1}{8}$ inches in diameter and $1\frac{1}{16}$ inches in height. (Fig. 8.) The briquette is allowed to dry in the air for approximately 18 hours and is then oven dried to constant weight at a temperature not exceeding 100° C. (212° F.). On being removed from the oven it is placed in a desiccator until cool and then tested by means of the apparatus shown in Figure 8. The slaking value is taken as the time required for the briquette to slake sufficiently to fall through the brass ring. Two specimens are made on each soil and the average considered as the slaking value of the soil.

Note.—*The briquettes are made $1\frac{1}{8}$ inches in diameter and $1\frac{1}{16}$ inches in height in order to produce, after shrinkage takes place, a briquette approximately 1 inch in diameter and 1 inch high. It has been found that approximately 35 grams of the soil and water mixture are required to produce a briquette of the desired size. The actual weight can only be determined by trial.*

DYE ADSORPTION TEST

It has been found that soils which appear to be practically identical, on mechanical analysis can give entirely different results for the moisture and volumetric tests. This indicates that the character of the material, especially of the clay, as well as its grading, has an important influence upon the physical characteristics of the soil. In order to determine the relative character of different soils, Dr. E. C. E. Lord, of the Bureau of Public Roads, has devised the following test:

A small sample of soil (about 10 grams), prepared as described under "Preparation of Soil Samples," paragraph 2, is oven dried to constant weight at a temperature not exceeding 100° C. (212° F.) and allowed to cool in a desiccator. The actual weight of soil used in this test is determined by the character of the soil, 0.5 gram being used for soils low in clay and 0.2 gram for those which are high in clay. Should it be difficult to judge the amount of clay, it is advisable to make a trial test. After cooling, a sample of the proper size, as determined above, is carefully weighed and transferred, by means of a camel's-hair

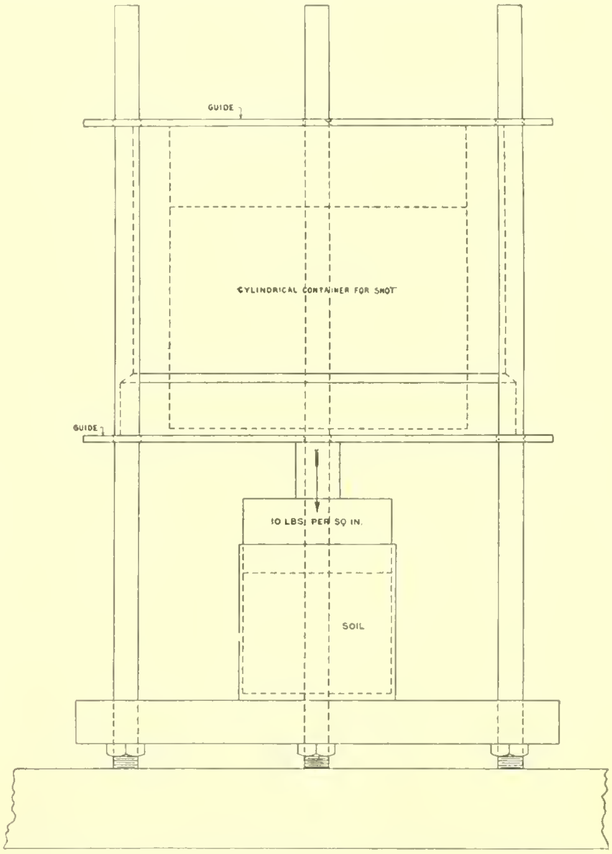

Fig. 6.—Apparatus for applying initial pressure to sample for comparative bearing value test

brush and glass funnel, into a separatory tube of about 5 cubic centimeters capacity which contains approximately 1 cubic centimeter of the dye solution. This mixture is thoroughly stirred with a polished glass rod for the purpose of coagulating the soil particles, causing them to settle out of the liquid. Stirring is continued until no soil particles are left in suspension, more dye solution being added, if necessary, to accomplish this result. The coagulated mixture is then introduced into a filter tube by means of the separatory funnel. (Fig. 9.) The filter tube should have been previously capped with filter paper and water allowed to run through it in order to prevent the trapping of air. The amount of water thus used should be determined in order that a correction may be made for it at the completion of the test. After the dye solution used for coagulation purposes has drained through the soil, additional dye should be added. Filtration should be allowed to continue until a marked discoloration appears in the filtrate. This indicates the end of the test, and the volume, in cubic centimeters of colorless filtrate, should be recorded. This volume less the cubic centimeters of water used in washing the filter tube, when converted to a basis of 1 gram of soil, represents the dye absorption number of the soil.

NOTE: *The dye employed in this test is a solution of 1 part of basic analine dye (methyl violet) in 1,000 parts of distilled water. The solution should be prepared in quantities of about 1 liter in order that a uniform solution may be provided for a large number of tests. Light and age materially affect the character of the dye solution, which consequently should be protected from strong light and should never be used when over 2 months old.*

INTERPRETATION OF TEST RESULTS

The interpretation of the results of tests on subgrade materials must, necessarily, be very general until sufficient field data can be obtained to make possible an accurate correlation of the test results with the observed behavior of the soil in the field. The limited research which has been given to the problem to date has yielded some very interesting conclusions, however, and it is confidently expected that, as more data are obtained, it will ultimately be possible to predict the field behavior of subgrade soils from an interpretation of the physical characteristics of the soils, as determined by laboratory tests.

The following comments should prove helpful in interpreting the results of tests on subgrade materials. It is advisable, however, that each investigator interpret the results in the light of his own experience, until sufficient data have been collected to permit the formulation of specific conclusions relative to the significance of the tests.

Mechanical analysis test.—The mechanical analysis is for the purpose of determining the gradation of the soil; i. e., the relative amounts of sand, silt, and clay contained in the soil. By itself, this test is not significant except in a very general way. It is known, of course, that soils containing a large percentage of clay (roughly, above 30 per cent) are, in general, undesirable, while those containing a small percentage (roughly, less than 30 per cent), as a rule, make good subgrades.

Moisture equivalent test.—This is one of the most important of the subgrade tests. It provides the means of comparing directly the relative ease with which subgrades of different characteristics may be drained. Soils that have high moisture equivalents are relatively more difficult to drain than those which have low moisture equivalents.

The deduction drawn from a recent investigation conducted by the Bureau of Public Roads in the Pacific Northwest was that soils that have moisture equivalents above 20 are undesirable as subgrade materials. Although this investigation was confined

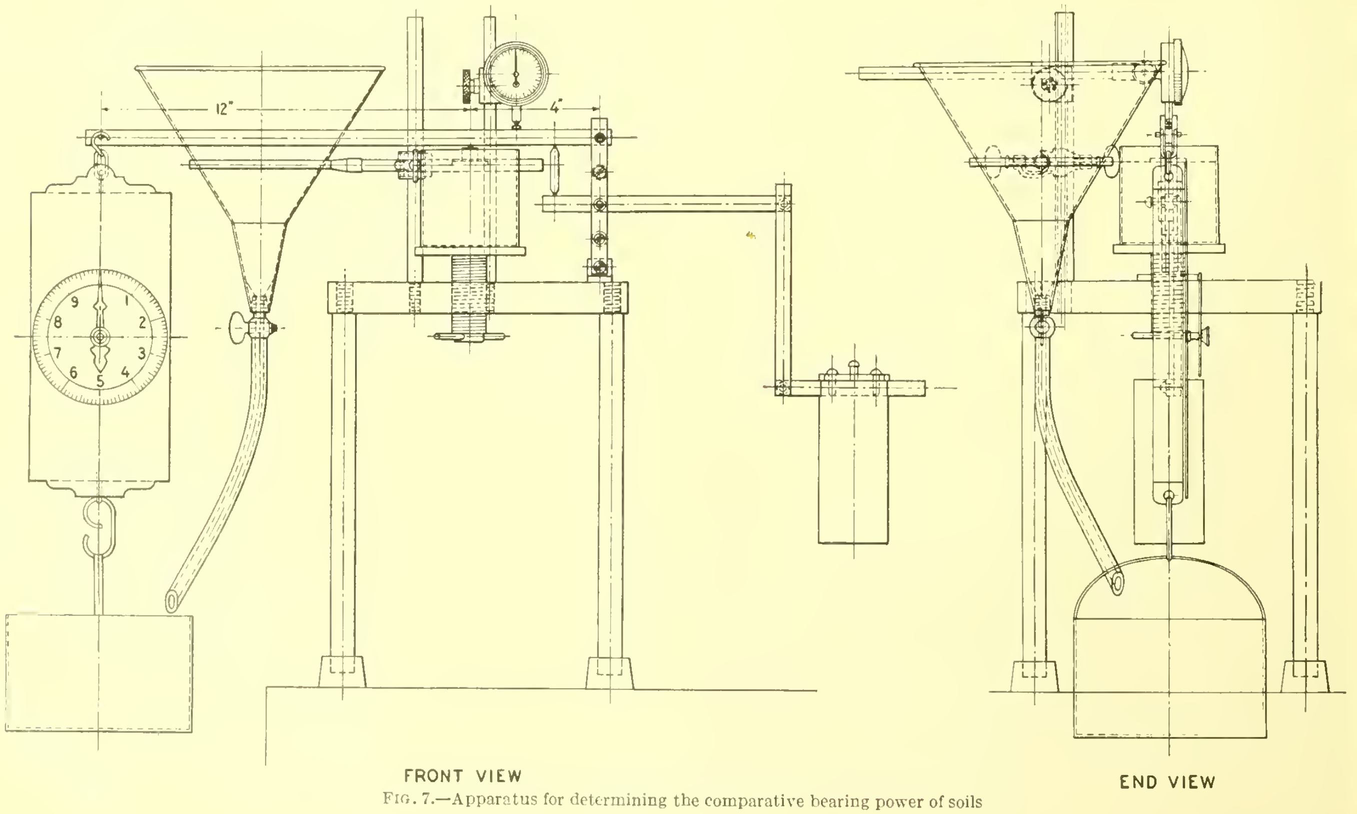

FIG. 7.—Apparatus for determining the comparative bearing power of soils

to a small section of the country, it is felt that the value of 20 is approximately correct as the line of demarcation between good and bad subgrade soils.

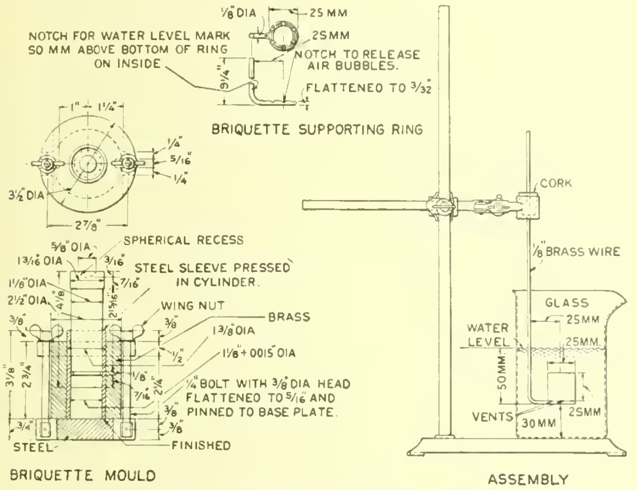

Fig. 8.—Apparatus for determining the slaking value of subgrade soils

Capillary moisture tests.—An effort is now being made to correlate the values obtained from this test with maximum moisture conditions existing in the field. Data so far collected seem to indicate that the maximum field moisture percentage of a soil is somewhat above the capillary moisture percentage as determined in the laboratory.

Volumetric shrinkage test.—The volumetric shrinkage of a subgrade soil should be determined under maximum field moisture conditions. Until more definite information is obtained, the capillary moisture as determined in the laboratory will be considered as approximating the maximum field moisture and the volume change test will be made under this moisture condition. Available information seems to indicate that subgrade soils having a percentage of volumetric shrinkage greater than 15 per cent will not make desirable subgrades.

Comparative bearing value test—Up to the present time it has been impossible to correlate this test with any field data so far obtained. Consequently this test should be considered only as a laboratory method for determining the comparative bearing value of subgrade soils.

Slaking value test.—The slaking value test should give information relative to the ability of subgrade soils to withstand the slaking action of water. Owing to the fact that this test has only been recently standardized it is not possible to give any limiting values at this time.

Dye adsorption test.—The dye adsorption test gives an indication of the character of the clay existing in the subgrade. It is quite possible to obtain approximately similar mechanical analyses from two different subgrade soils, and yet these soils may react in an entirely different manner to the other physical tests. This is due, undoubtedly, to the difference in character of the constituent parts of the soil, especially the clay. A soil having a large percentage of very active clay will give an adsorption number higher than that obtained from a soil having approximately the same percentage of less active clay. High percentages of clay accompanied by high adsorption numbers almost invariably indicate poor subgrade material.

In general it may be said then that soils having a high moisture equivalent, a high capillary moisture percentage, a high volumetric shrinkage, and a high dye adsorption number will be very poor as a subgrade material, while if these values are low the material may be expected to give satisfactory service.

(Continued on page 41)

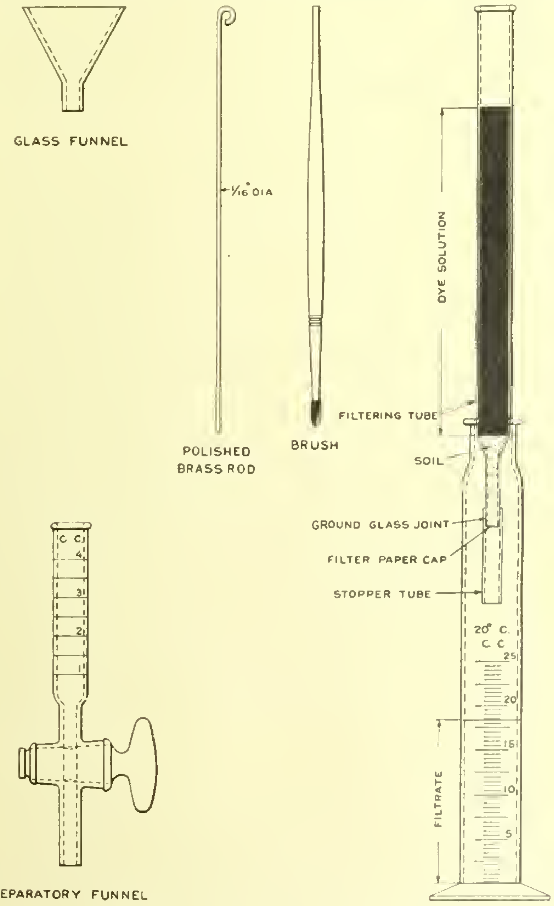

Fig. 9.—Apparatus for determining the dye adsorption of subgrade soils

EXPLANATION OF CHART

Subgrade soils are divided into six classes, according to the relative amounts of sand, silt, and clay which exist in them, the name of the class being determined by the predominating fraction and modified by the next predominating fraction as obtained from the mechanical analysis. This gives six classes as follows: Silty clay, sandy silt, sandy clay, clayey sand, clayey silt, silty clay.

The following example illustrates the use of the chart.

The mechanical analysis of a subgrade soil is as follows: Sand, 20 per cent; silt, 30 per cent; and clay, 50 per cent. This soil would be classified as a silty clay and plotted on the chart as shown.

DEFINITIONS

Sand is that material which passes a 10-mesh sieve, is retained on the 200-mesh sieve and subsides through a height of 8 cm. in ammoniated water (concentration 1 to 500) in 8 minutes.

Silt is that material which passes a 200-mesh sieve and subsides through a height of 8 cm. in ammoniated water (concentration 1 to 500) in 8 minutes.

Clay is that material which passes a 200-mesh sieve and which does not subside through a height of 8 cm. in ammoniated water (concentration 1 to 500) in 8 minutes.

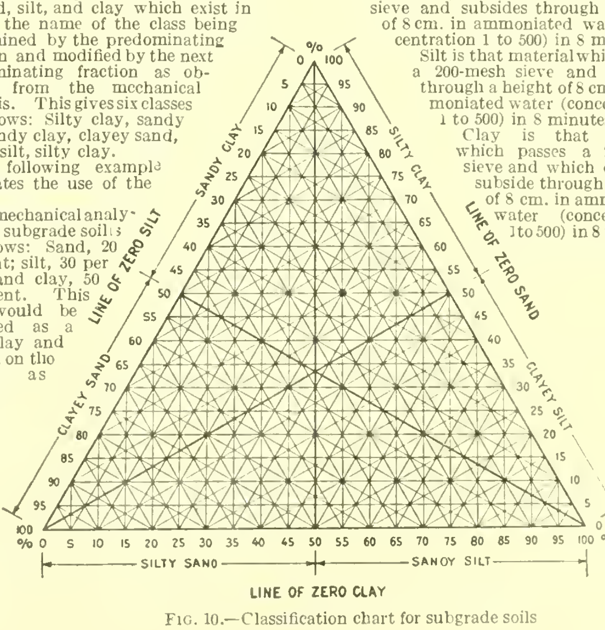

Fig. 10.—Classification chart for subgrade soils

EFFECT OF CAPPING ON STRENGTH OF CORES DRILLED FROM CONCRETE PAVEMENTS

By R. E. BERGFORD, Assistant Engineer of Tests, Minnesota Highway Department

IN MAKING compression tests of concrete cylinders or of specimens of concrete drilled from actual structures, the importance of making the proper correction for the variation in the height of the cylinder relative to its diameter is a factor which has possibly not been sufficiently considered in the past. The necessity for definite information on this question has been felt by the writer for some time in connection with the tests made on cores drilled from pavements in Minnesota.

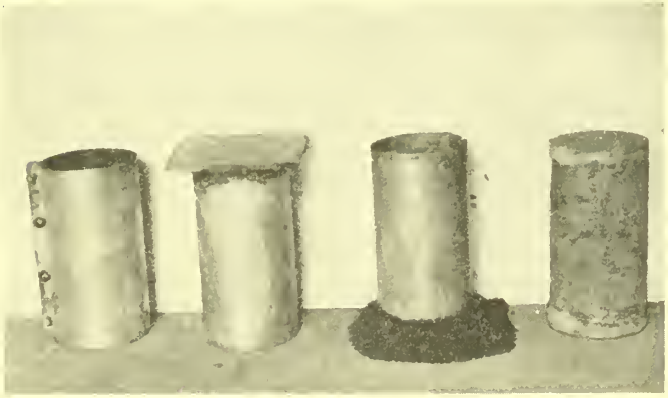

FIG. 1.—Illustrating the method of capping cores.

The diameter of the test specimen drilled is 4.35 inches and the height varies according to the thickness of the slab inspected. Cores are drilled not only from pavements constructed by the State but also from pavements constructed by various municipalities under a great variety of specifications. As is to be expected there is a considerable range in height of cores, heights varying from 3 to 10 inches.

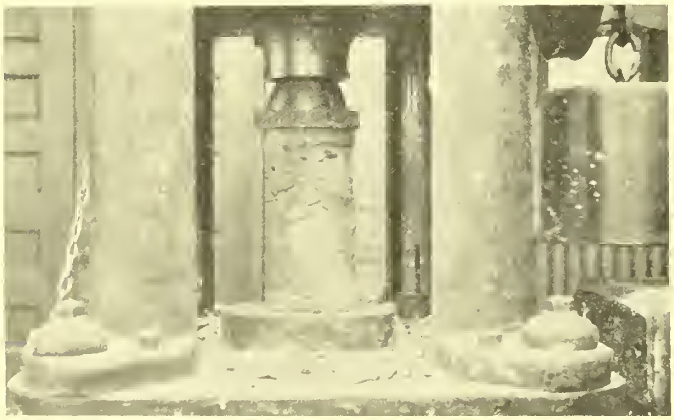

FIG. 2.—Showing core in place ready to be tested.

Owing to the great range in height of the test specimens, as well as to the more or less irregular surface on the end of the core which is in contact with the subgrade, the problem presented is, first, the best and most economical method of preparing the cores for the compression test, and second, the interpretation of results of compression tests on these cores.

The most satisfactory method of preparing the cores for a compression test would probably be to saw or grind both ends to a square and true surface. As this method necessitates the use of more or less elaborate machinery, the simpler method of capping the ends of the cores with a rich cement mortar has been adopted. The following procedure is used in capping the cores.

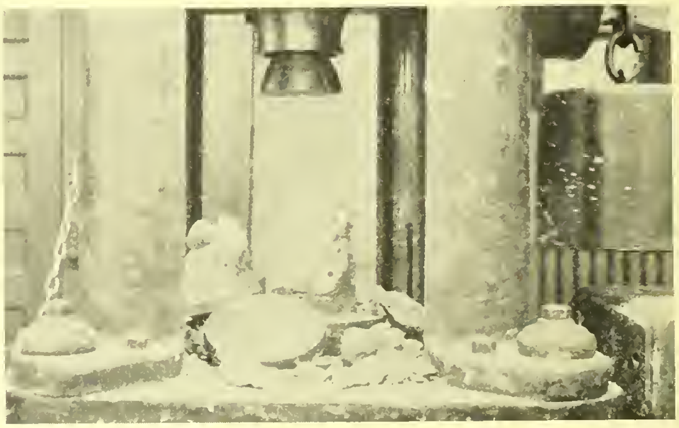

FIG. 3.—Showing core after testing.

METHOD OF CAPPING CORES

The end surfaces of the cores are first thoroughly brushed and washed in order to remove any dirt or loose material which might be adhering to the surface. The specimen is then placed in a galvanized iron form 4.35 inches in diameter and 8.7 inches high and centered accurately so as to give an equal thickness to the cap on each end.

The mortar used to cap the ends consists of a mixture of 4 parts of Portland cement to 1 part of sand by volume and enough water to make the mortar workable.

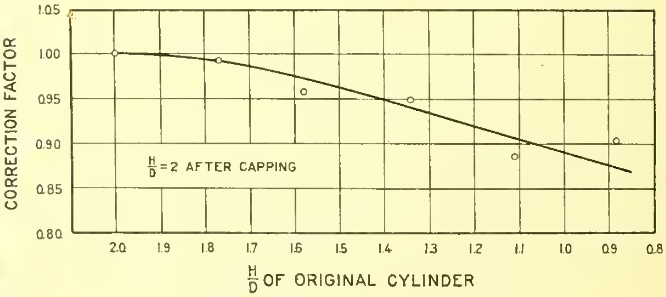

FIG. 4.—Effect of capping on test cylinders.

This mixture when properly cured is found to give a compressive strength greater than that of any of the cores. The capping material is placed in the form, tamped, rounded off with a trowel, and flattened off with a glass plate which is left in position for 24 hours. Damp sand is placed around the edge of the cap and left there for 24 hours in order to prevent checking or cracking of the mortar due to excessive shrinkage. At the end of 24 hours the glass plate is removed and the opposite end capped in a like manner. After both ends of the core have been capped it is placed in a moist curing room and remains there until tested. The method of capping is illustrated in Figure 1. The method of testing the cores is illustrated in Figures 2 and 3.

The question naturally has arisen as to the correct interpretation of the test results. Very little data are available in regard to correction factors to be applied in the case of compression tests on cylinders of various heights. The recommended shape for a cylinder is one having a diameter equal to one-half its height, i. e., $\frac{H}{D}=2$. The cores as capped in the laboratory have a ratio of $\frac{H}{D}=2$ after they are capped. The variable is introduced by the fact that the capping material is not the same as the concrete composing the core.

In order that we might be able to determine the correct value for the compressive strength of cores thus tested, a series of concrete cylinders was made up, the diameter of each cylinder being 4.35 inches, and the heights ranging from 3.7 to 8.7 inches. The cylinders which are 8.7 inches high were taken as standard. The remaining cylinders were capped in the manner previously described so that the height after capping was equal to twice the diameter. The mix used in making the test specimens was 1:2:4 by volume. Four different aggregates, including sandstone, trap, gravel, and limestone, were used. The results of this investigation are shown in Figure 4, which shows the correction value to be applied to cores of varying heights. Each point in this graph represents the average of tests on 12 cores.

FIFTH INTERNATIONAL ROAD CONGRESS ANNOUNCED

The Fifth International Road Congress will meet at Milan, Italy, at the invitation of the Italian Government, from September 6 to 13, 1926.

The program announced for the congress includes the discussion of the following questions:

First Section.—Construction and Maintenance

First question.—Concrete roads.
  Progress achieved in the use of materials for the construction of roads in cement-concrete.
Second question.—Bituminous and asphaltic roads.
  Qualities required of the materials employed: Binder; aggregate.
Third question.—Standardization of tests for the following: Roads—materials.
  Coal tar, bitumens, and asphalts.

Second Section.—Traffic and Administration

Fourth question.—Census of traffic.
  Search for uniform international bases for adoption in every country.
Fifth question.—Development and planning of towns in interests of traffic.
  Progress achieved in the general control of traffic in towns.
Sixth question.—Special roads reserved for motor traffic.
  What conditions justify their being built.
  The appropriate authorities for their initiation and construction.
  Financial arrangements: Contribution from public funds; tolls; rules for traffic circulation and for exploitation.
  Relationship and connection between motor roads and other highways in the interests of safety and the continuity of traffic generally.

An international exposition open to manufacturers or producers of materials and implements used in the construction and maintenance of roads, as well as to manufacturers of vehicles and accessories, will be held in connection with the congress.

The Italian committee of organization, in cooperation with the executive bureau of the Permanent International Association of Road Congresses, has arranged, in addition to the regular sessions, for a series of visits to modern roads built or under construction, notably the Autodrome of Monza, where a race for the grand prize will be run, and the new automobile highways which connect Milan with the Italian lakes. Visits are also scheduled to industrial plants and road-construction projects which will be under way at the time.

Information concerning the exposition may be obtained from the Secretariat de la Commission Italienne d'organisation du Veme Congres international de la Route, Via Sala 3, Milan (M. l'Ingenisur G. Lori, Secretaire General), or the Secretariat de l'Association Internationale Permanente des Congres de la Route, 1 Avenue d'Iena, Paris.

Details of the arrangements for the congress and the time schedule, as well as directions in regard to travel and lodging, will be published later.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

MARCH 31, 1925

GOVERNMENT PRINTING OFFICE

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1925									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1924			PROJECTS COMPLETED SINCE JUNE 30, 1924			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	4,698,721.63	2.186,247.54	464.1	1,211,826.12	697,175.34	132.2	16,535,183.95	7,536,992.14	819.8	83,897.90	41,948.96	0.4	2,432,469.03	Alabama
Arizona	8.338,366.41	4,287,683.88	527.8	860,758.13	490,731.10	62.0	1.912,560.69	1,159,156.39	154.4	580,917.91	354,999.13	41.9	2,259,301.50	Arizona
Arkansas	11,094,761.31	4,424,345.63	944.4	1,826,278.21	782,661.32	78.9	6,842,562.82	2,888,860.18	371.1	439,930.06	175,800.66	24.1	2,054,896.21	Arkansas
California	12,999,076.03	6.647,148.17	533.7	7,728,789.52	4.173,069.86	308.4	9,570,563.27	4,622,869.53	240.2				6,122,854.44	California
Colorado	8.108,070.31	4,029,898.97	502.6	2,043,485.48	1.088,727.79	91.4	4,812,896.07	2,557,822.76	187.2	434,324.88	156,617.34	9.9	3,100,151.14	Colorado
Connecticut	3,062,872.02	1,269,558.60	73.6	344,021.87	163,383.00	8.2	3,114,792.92	980,844.04	50.8				1,442,210.36	Connecticut
Delaware	3,056,832.22	1,007,714.83	72.5	951,730.50	380,775.82	27.4	786,048.44	321,281.10	19.5				395,383.25	Delaware
Florida	961,134.07	461,470.92	48.8	1,903,234.92	924,017.06	43.4	8,936,479.45	4,276,067.21	240.8	116,547.09	91,547.09	0.1	1,426,662.73	Florida
Georgia	17,167,373.32	7,956,805.20	1,214.2	2,505,073.82	1,220,161.94	229.4	10,868,886.42	5,246,583.68	739.0	662,027.30	202,661.28	30.9	1,807,773.90	Georgia
Idaho	8,181,697.92	4,092,396.52	506.8	1,026,866.02	584,588.92	68.4	1,884,090.99	1,160,875.03	129.9	772,764.17	525,416.88	62.2	1,251,363.66	Idaho
Illinois	26,964,706.06	12,279,546.33	804.7	9,498,957.56	4,649,408.31	307.6	7,989,434.22	3,887,002.14	273.5	2,671,148.06	1,322,589.49	96.8	4,489,424.73	Illinois
Indiana	7,677,444.16	3,655,540.97	225.7	4,612,589.31	2,222,109.13	152.9	12,569,653.34	6,136,323.77	406.2				4,237,111.13	Indiana
Iowa	23,196,778.19	9,237,031.86	1,682.9	3,319,941.92	1,516,911.81	238.1	5,746,438.71	2,635,611.59	429.3	2,920,333.58	1,231,126.05	112.2	2,786,851.69	Iowa
Kansas	17,084,136.48	6,043,176.80	502.7	6,930,776.77	2,750,857.38	244.2	14,746,138.35	6,033,891.84	642.9	1,310,537.95	453,819.17	45.6	2,091,903.81	Kansas
Kentucky	10,822,980.31	4,613,947.28	429.4	2,860,912.31	1,134,921.19	110.1	8,098,252.26	3,687,766.78	307.6	406,326.77	140,280.00		2,206,440.76	Kentucky
Louisiana	8,488,463.18	3,636,143.36	661.2	2,041,083.47	993,840.89	172.1	4,361,756.39	2,172,989.90	240.9	14,353.66	7,176.82		1,452,653.03	Louisiana
Maine	6,911,058.78	3,299,935.38	230.7	1,263,222.53	607,934.96	50.7	749,878.17	360,363.39	24.2				1,616,878.28	Maine
Maryland	6,760,044.42	3,213,321.78	243.2	646,410.32	307,486.47	26.6	2,464,768.03	1,066,404.17	76.2	337,942.11	122,411.22	8.8	676,109.36	Maryland
Massachusetts	10,191,202.02	4,105,727.22	232.8	2,128,343.98	788,018.34	39.7	4,794,626.96	1,430,934.20	74.4	1,234,606.08	354,646.77	21.4	2,330,571.47	Massachusetts
Michigan	13,434,135.07	6,060,612.23	494.5	2,165,559.55	994,055.92	101.6	14,965,650.26	7,044,686.58	479.4	163,170.00	81,585.00	6.4	3,924,059.27	Michigan
Minnesota	24,037,561.24	9,885,843.07	2,292.0	6,378,124.65	2,852,798.97	429.2	6,294,034.56	2,923,600.00	662.5	679,082.36	46,300.00	67.8	1,734,027.96	Minnesota
Mississippi	7,888,193.89	3,828,941.39	655.0	1,969,486.66	945,691.68	141.4	8,320,046.33	4,180,767.52	486.0	77,107.38	36,968.23	4.5	1,830,964.18	Mississippi
Missouri	11,352,027.70	6,245,899.18	803.5	4,226,155.25	2,092,916.66	242.8	23,815,757.38	9,747,509.61	784.4	4,593,722.03	1,173,042.31	164.6	2,098,547.34	Missouri
Montana	8,867,279.16	4,384,335.12	791.4	1,289,321.25	933,188.03	130.2	1,885,734.50	1,380,489.53	161.4	1,038,200.22	690,707.66	117.2	6,226,169.66	Montana
Nebraska	7,876,337.16	3,714,691.59	1,440.4	791,712.26	381,242.14	60.4	8,066,148.46	3,952,701.66	874.1	413,267.76	206,633.87	46.6	4,777,646.74	Nebraska
Nevada	3,460,245.52	1,853,624.98	225.6	1,446,557.71	1,228,226.07	131.4	4,300,066.51	3,622,200.69	351.6	155,693.56	131,383.28	10.8	1,002,962.98	Nevada
New Hampshire	3,076,750.19	1,487,867.58	171.3	1,052,452.40	481,514.29	35.7	778,364.18	372,998.07	25.1	60,651.77	29,200.26	2.6	429,008.81	New Hampshire
New Jersey	7,623,795.12	2,661,531.49	148.7	3,757,997.15	991,122.25	59.6	7,267,004.53	2,288,662.36	41.5	303,401.87	88,599.00	6.9	1,494,513.91	New Jersey
New Mexico	6,306,286.45	2,758,849.68	714.3	2,167,396.87	1,377,821.90	224.2	6,663,914.48	3,710,640.73	639.6	250,983.39	154,354.77	16.7	1,772,830.92	New Mexico
New York	18,862,742.49	8,257,844.44	572.7	4,371,833.80	1,810,063.42	121.1	27,762,574.84	10,358,069.38	612.7	6,455,088.00	1,870,970.00	121.8	8,068,296.76	New York
North Carolina	12,567,732.97	5,676,757.66	884.7	6,093,681.22	2,189,158.07	169.8	7,502,036.12	3,037,940.48	206.2	1,791,845.30	861,774.46	66.7	2,237,788.33	North Carolina
North Dakota	9,088,973.11	4,418,505.42	1,587.9	1,707,266.90	833,931.18	325.6	2,486,235.72	1,248,727.03	347.1	679,726.71	289,863.29	72.2	2,753,328.08	North Dakota
Ohio	33,122,751.43	11,879,917.99	962.5	7,596,806.64	3,099,876.94	207.1	8,614,974.91	2,964,707.50	250.2	2,784,105.61	1,049,339.16	80.6	3,936,911.42	Ohio
Oklahoma	12,986,865.26	5,888,852.03	497.3	5,699,897.38	2,765,414.67	246.0	6,386,181.94	3,054,129.10	295.5	2,033,937.26	913,789.21	79.2	1,669,622.99	Oklahoma
Oregon	12,082,873.17	6,819,093.79	656.6	2,162,545.40	1,274,243.08	140.2	2,409,677.77	1,386,773.92	105.7				1,206,716.21	Oregon
Pennsylvania	36,825,248.98	14,114,694.79	729.7	3,160,057.54	1,190,520.67	66.1	21,710,921.89	6,724,702.50	354.4	12,532,653.38	4,473,268.00	301.8	2,458,553.14	Pennsylvania
Rhode Island	1,774,397.26	779,227.96	46.0	854,098.96	340,460.13	18.8	1,171,551.79	341,168.84	15.8	714,691.45	197,520.75	11.9	640,288.32	Rhode Island
South Carolina	9,016,476.73	4,124,045.22	924.4	1,822,899.07	846,189.44	280.6	6,094,619.84	2,293,646.96	391.6	965,012.37	261,412.93	19.9	1,214,800.46	South Carolina
South Dakota	8,674,597.86	4,244,636.27	989.8	2,874,673.89	1,498,821.92	429.1	6,169,362.95	3,009,035.89	798.9	471,109.62	236,654.79	106.7	945,661.13	South Dakota
Tennessee	6,805,683.35	3,313,936.07	259.6	6,721,124.74	2,802,118.58	196.2	11,996,215.60	5,467,986.85	430.6	656,537.10	323,649.70	33.8	1,749,931.80	Tennessee
Texas	42,341,998.56	15,190,624.91	3,122.8	8,756,960.46	3,580,315.53	546.7	23,062,112.73	9,584,555.77	1,422.7	4,977,485.76	2,152,613.76	329.4	4,631,818.03	Texas
Utah	3,304,423.75	1,895,805.92	219.0	1,696,373.90	1,028,206.53	147.3	3,914,748.38	2,560,601.77	225.7	318,700.99	219,171.60	31.6	1,259,154.28	Utah
Vermont	1,922,114.16	942,769.12	74.4	641,963.66	308,451.42	19.2	1,632,633.82	776,217.63	40.0	10,699.63	6,349.76		866,816.07	Vermont
Virginia	10,035,301.48	4,801,782.43	562.5	2,368,636.37	1,136,082.48	90.2	9,437,249.00	4,330,933.38	324.1	1,683,159.96	703,141.78	62.8	1,070,726.93	Virginia
Washington	11,384,615.67	6,290,895.45	457.0	1,172,750.16	611,294.59	61.9	3,367,073.68	1,599,900.00	134.9	296,296.71	134,000.00	9.0	1,469,674.96	Washington
West Virginia	6,489,747.95	2,365,041.63	255.6	1,853,452.91	865,261.80	71.1	4,166,291.65	1,791,537.78	127.9	2,458,182.33	900,746.56	63.9	628,850.33	West Virginia
Wisconsin	18,753,903.16	7,441,033.67	1,326.3	3,001,359.02	1,452,667.68	121.0	2,317,433.76	1,096,311.36	109.7	53,417.25	26,708.00	3.9	5,535,038.39	Wisconsin
Wyoming	6,127,625.61	3,078,098.70	687.6	2,629,027.49	1,626,626.97	290.6	2,736,020.87	1,702,071.25	122.2	121,396.44	78,481.00	9.1	1,137,020.08	Wyoming
Hawaii										$ 342,277.22	$ 97,440.00	6.6	$ 633,810.00	Hawaii
TOTALS	$ 549,655,391.27	$ 237,852,399.82	34,452.9	$ 143,022,372.98	$ 66,814,951.42	7,486.5	$ 359,976,438.80	$ 159,694,802.96	16,148.2	58,867,259.82	22,504,507.86	2,284.9	111,383,337.94	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 96,361,146.96 Federal aid $ 41,277,405.13 Miles 3,759.9

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Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

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* Department supply exhausted.

THE GASOLINE TAX IN 1924

States	Tax rate per gallon on Dec. 31, 1924	Gross receipts 1924	Amount applied to road work under supervision of State highway departments
	Cents		
Alabama	2	$1,738,661	----------
Arizona	3	730,838	$365,419
Arkansas	4	2,768,535	2,268,535
California [1]	2	11,993,222	5,996,611
Colorado	2	1,725,957	819,830
Connecticut [1]	1	978,283	978,283
Delaware	2	304,392	304,392
Florida	3	3,658,677	2,575,199
Georgia	3	4,527,471	1,509,157
Idaho	2	545,672	545,672
Illinois	----------	(No tax)	----------
Indiana	2	4,925,372	4,187,855
Iowa	----------	(No tax)	----------
Kansas	(2) -	(No tax)	----------
Kentucky	3	1,660,938	1,660,938
Louisiana	2	1,335,320	1,335,320
Maine	1	522,250	522,250
Maryland	2	1,588,422	1,111,895
Massachusetts	----------	(No tax)	----------
Michigan	(2) -	(No tax)	----------
Minnesota	----------	(No tax)	----------
Mississippi [1]	3	1,648,748	787,319
Missouri	----------	(No tax)	----------
Montana	2	619,295	123,859
Nebraska	----------	(No tax)	----------
Nevada	2	162,596	60,000
New Hampshire	2	587,845	587,845
New Jersey	----------	(No tax)	----------
New Mexico	1	194,983	185,234
New York	----------	(No tax)	----------
North Carolina	[2] 3	4,529,048	[3] 4,520,000
North Dakota	1	442,969	----------
Ohio	----------	(No tax)	----------
Oklahoma	2.5	2,983,501	1,544,600
Oregon	3	2,698,778	2,582,890
Pennsylvania	2	9,089,541	----------
Rhode Island	----------	(No tax)	----------
South Carolina [1]	3	2,186,137	728,889
South Dakota	2	1,205,155	1,106,635
Tennessee	[2] 2	1,812,235	1,812,235
Texas	1	3,892,769	2,919,577
Utah [1]	2.5	684,361	682,985
Vermont [1]	1	230,865	230,865
Virginia	3	3,313,188	[3] 2,208,571
Washington	2	2,635,411	2,635,411
West Virginia	2	1,231,944	1,231,944
Wisconsin	----------	(No tax)	----------
Wyoming	[2] 1	200,319	200,319
District of Columbia	2	380,792	380,792
Totals	----------	$79,734,490	$48,711,792

[1] Data given cover calendar years, except for the following States, where fiscal years end as shown: California, Jan. 31; Connecticut, June 30; Mississippi, South Carolina, Utah and Vermont, Nov. 30.

[2] To date in 1925, these States have enacted new gasoline tax rates effective as follows: Kansas, 2 cents (May 1); Michigan 2 cents (Feb. 15); North Carolina, 4 cents (Mar. 5); Tennessee, 3 cents (Feb. 9); Wyoming, 2½ cents (Mar.).

[3] Approximate.

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 6, NO. 3 ▽ MAY, 1925

Approach to Observation Station on Maine Highway Transportation Survey

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1925

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH
U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

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THE MAINE HIGHWAY TRANSPORTATION SURVEY

A PRELIMINARY REPORT

BY THE DIVISION OF HIGHWAY TRANSPORT AND ECONOMICS, U. S. BUREAU OF PUBLIC ROADS

Reported by J. G. McKAY, Chief of Division, and O. M. ELVEHJEM, Highway Economist

A PRELIMINARY analysis of the evidence obtained in the Maine highway transportation survey, conducted by the United States Bureau of Public Roads in cooperation with the Maine State Highway Commission has developed a number of interesting and useful facts with regard to the traffic on the Maine highways, its growth over a period of years, present distribution over the State system, probable future density, and other matters of interest generally to all concerned with the planning and construction of highways. The evidence analyzed was recorded in the course of the field study which was begun July 1, 1924, and lasted until October 31, 1924.

The rapid increase in the demand for highway service is indicated by the increase in motor vehicle registration and in the traffic on the State highway system from 1916 to 1924. Between 1916 and 1920 the registration doubled, it doubled again from 1920 to 1924, and it is estimated that it will double again from 1924 to 1930. Parelleling this increase, the traffic doubled between 1916 and 1919, it doubled again from 1919 to 1923, and it is estimated that it will double again in the period from 1924 to 1930. But while the demand for highway service in the State, considering the transportation of passengers and commodities together, has thus apparently doubled and redoubled in the past eight years, consideration of the motor-truck and passenger-car traffic separately suggests that there is a difference, so far as the State highway system is concerned, in the demands for motor-truck and passenger-car transportation. This is evidenced by the fact that truck traffic has increased at a slower rate than truck registration, while passenger-car traffic has increased at a faster rate than passenger-car registration; but, for total vehicles, the rates of increase of highway traffic and vehicle registration have been nearly equal.

One of the most valuable results of the survey is the collection of data upon which future traffic may be forecast with reasonable accuracy. This makes possible the development of a definite program of future improvement by determining the routes to be improved, the order of their improvement, and the type of improvement required. The forecast of future traffic is of particular value in Maine since the State has reached the second critical stage in the development of its highways. Hitherto the highway commission has wisely constructed large mileages of gravel roads to make accessible the greatest possible area of the State with the funds available.

The concentration of motor-vehicle traffic around the centers of population on the principal State roads now makes necessary a definite improvement policy governing the selection of the routes to be reconstructed with surfaces of higher type, and the determination of the type of surfacing. The Maine experience indicates that a gravel road will not successfully carry over 500 vehicles per 12-hour day without resorting to surface treatment.

Application of the most reliable available information with regard to the savings in operating costs of vehicles made possible by improvement in road surfaces indicates that, on the basis of present traffic, the 300 miles of most heavily traveled roads in the State could be improved from an earth-road condition to a condition in which every mile would be surfaced with concrete, and the entire cost of the improvement, with interest at 4 per cent, would be repaid by the savings in operating costs of passenger cars only in slightly over four years.

SEVEN PER CENT OF THE ROADS SERVE MORE THAN HALF THE TRAFFIC

The survey has brought to light a number of interesting facts with regard to the traffic on the roads of the State. For example, it is shown that the primary highway system which embraces only 7.1 per cent of the total highway mileage carries 53.4 per cent of the total daily vehicle mileage. Furthermore, 18.4 per cent of the primary system carries 38.7 per cent of the total daily vehicle mileage on the system. From this it follows that, with respect to the entire highway system of the State, 1.3 per cent of the total mileage serves more than a fifth of the traffic, as measured in vehicle miles.

It is evident that the heavy concentration of traffic is confined to a relatively small percentage of the total highway mileage. For this reason it is advocated that traffic zones should be created to bring together for construction and maintenance purposes those sections of the highway system which serve approximately the same amount and type of traffic.

The traffic importance of the primary system as compared with the secondary system appears even greater when considered from the point of view of motor-truck traffic than when considered from the standpoint of passenger-car traffic. Practically all trucks using the Maine highways have capacities between one-half and 2½ tons, and the number of trucks of 5 tons capacity or over is practically negligible. Over 80 per cent of the trucks observed were equipped with pneumatic tires, and from 55 to 67 per cent were loaded. Wheel loads in excess of 2,500 pounds were found to be very exceptional on trucks weighing less than 6,000 pounds gross, and the maximum wheel load for trucks of less than 12,000 pounds gross weight (3 tons capacity) was found to be 5,000 pounds. Over 98 per cent of such trucks, however, have wheel loads less than 4,500 pounds.

On a vehicle-mileage basis it is found that a considerable portion of the cost of providing highway service on the primary system is due to its use by foreign vehicles. But these vehicles pay into the State treasury through the gasoline tax a sum which the State would not receive if there were no gasoline tax, and the amount of the tax paid is proportional to their use of the system. The use of the State's roads by foreign motor trucks is much less extensive than its use by foreign passenger cars and, except near the State line and on a few major highways, is negligible.

As a result of the survey a forecast traffic map has been prepared which shows the anticipated density of traffic on roads of the State's primary and secondary systems between July and November, 1930. Neglecting such factors as the effect of major mechanical improvements of automobiles, it is believed that the actual traffic in 1930 will closely approximate the estimated traffic as recorded on this map, and the map has therefore been used as a basis for a number of detailed suggestions with regard to the program of highway improvement up to 1930.

TRAFFIC ON PRIMARY, SECONDARY, AND THIRD-CLASS HIGHWAYS

There are 23,104 miles of highway in the State. Of this mileage 1,630 miles, constituting the State highway system, is defined as the primary system. State-aid highways, consisting of 4,049 miles not included in the State highway system but serving as feeders to it, are defined as the secondary system. Third-class roads, comprising 17,425 miles, include all highways not included in the State or State-aid systems.[1]

This classification of the roads, it was one of the purposes of the survey to check by a more exact determination of the traffic served by them. At the same time the survey methods were designed to supply the information needed to decide upon the adequacy of the types of surfaces laid on roads of the three systems with respect to present and probable future traffic, and to serve as a basis for the equitable partition of funds available for construction and maintenance.

To serve these purposes it was necessary that the survey supply four general classes of data, as follows: (1) Density of traffic on all parts of each system; (2) the size and loading of motor truck traffic; (3) vehicle-mileage per year on each of the three systems; and, (4) the probable growth of traffic over a reasonable future period.

The traffic classification of roads and the selection of the most suitable type of highway surface to serve traffic depends upon the type of traffic units as well as the number of these units. The more important considerations are: (1) Density of present and future total traffic; (2) the ratio of trucks to total vehicles; (3) the proportion of trucks of large, medium, and small capacity, and the resulting gross loads; (4) the maximum wheel loads; and, (5) the frequency of critical gross loads and wheel loads. In individual cases other factors must also be considered, but in general the more important considerations are those above mentioned. The final selection of type of highway surface depends upon certain physical considerations, such as availability and cost of materials, as well as upon traffic considerations.

Vehicle mileage involving, as it does, the factors of number of vehicles and mileage traveled serves as the basis for the equitable allocation of funds in proportion to the utilization of the three systems; and the extension of the curves of traffic and vehicle registration makes possible a forecast of the growth of traffic which will enable those in charge of highway administration to make the necessary provision for future maintenance and construction.

Density of traffic on the three systems.—The density of traffic,[2] which is one of the criteria determining the classification of roads and the types of surface needed, has been computed from the number of vehicles passing each of the survey stations during the observation periods corrected for each station to a 24-hour day.[3]

Computed in this way the average density of traffic on the three systems, as now designated, is shown in Table 1. Considering each of the systems as a whole the table indicates that the average density of traffic on the primary system is thirty-six times as great as on the third-class system and over four times as great as on the secondary system; and, as shown in Table 2, this same relation applies approximately to the truck and passenger-car traffic separately as well as to the total traffic, from which it follows, also, that the relative density of motor-truck and passenger-car traffic on the three systems is approximately the same, the ratio in each case being about 1 to 10.

TABLE 1.—*Average density of traffic on the primary, secondary, and third-class highway systems, July 1 to October 31, 1924*

Highway system	Average density of traffic	Index of relative density of traffic (third class= 100 per cent)
	Vehicles per day	*Per cent*
Primary (1,630 miles)	1,044	3,600
Secondary (4,049 miles)	244	840
Third class (17,425 miles)	29	100

TABLE 2.—*Average density of motor-truck and passenger-car traffic on the primary, secondary, and third-class highway systems, July 1 to October 31, 1924*

Highway system	Average density of passenger-car traffic	Index of relative density of passenger-car traffic (third class=100 per cent)	Average density of motor-truck traffic	Index of relative density of motor-truck traffic (third class=100 per cent)	Ratio of passenger-car to motor-truck traffic
	Vehicles per day	*Per cent*	*Vehicles per day*	*Per cent*	
Primary (1,630 miles)	950	3,520	94	4,700	10.2
Secondary (4,049 miles)	221	820	23	1,150	9.6
Third class (17,425 miles)	27	100	2	100	13.5

Other things being equal, these indices describe the relative average highway requirements of the traffic on the three systems and govern the average expenditures which may justifiably be made for the improvement of each mile of each system. Thus, if the ratios of motor trucks to passenger cars is approximately the same, the fact that the average density of traffic on the primary system is thirty-six times as great as the average density on the third-class system means that the average justifiable expenditure for the improvement of each mile of the primary system is thirty-six times as great as the average expenditure which can be justified for each mile of the third-class system.

Traffic served by the three systems.—These indices also indicate the relative transportation service afforded by each mile of the three systems, but by reason of the different extent of the systems they do not describe the relative magnitude of the service rendered by the systems or their relative total utilization considered as parts of the whole system of the State. This can only be described in terms of the total daily vehicle mileage on the three systems, which is the sum of the distances traveled in a day by all vehicles on each system.[4]

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[1] Chap. 25, Sec. 5, Laws of Maine, 1917.

[2] In this report certain terms, frequently used, have invariably the same meaning. These terms and their definitions are as follows:

Vehicles refers only to motor vehicles (passenger cars and trucks) exclusive of horse-drawn conveyances.

Traffic is defined as the movement to and fro of vehicles over a highway.

Density of traffic is defined as the number of motor vehicles passing any given point on a highway in a unit of time. Unless a different unit of time is specifically stated it refers to the number of vehicles passing any given point on a highway during a day of 24 hours.

Daily refers to a day of 24 hours.

Vehicle-mile is defined as the movement of a vehicle 1 mile.

Vehicle-miles per mile is defined as the sum of the mileage traveled by all motor vehicles in passing over 1 mile of highway. It is numerically equivalent to the average density of traffic on the mile of highway.

[3] The accuracy of the determination of density of traffic is influenced by the distance between the survey stations. Exactness of method would require a density record for each point on the highway system where traffic varies. The cost involved in proportion to the relatively small gain in accuracy does not justify the location of recording stations at close intervals. The density computed for each station on the Maine highway system is applied to sections of the system reasonably adjacent to each station.

The importance of distinguishing between the total vehicle mileage on each system and the average density of traffic on each (which is numerically equivalent to the vehicle mileage per mile) is clearly indicated in Table 3. For, while the average density or vehicle mileage per mile on the primary system is thirty-six times the average density on the third-class system, Table 3 shows that the total traffic service rendered by the primary system, of 1,630 miles, as measured in vehicle-miles, is three and one-half times as great as the total traffic service rendered by the 17,425 miles of the third-class system. The latter relation should control the apportionment of available funds to the three systems; the former should control the justifiable expenditure per mile on each system.

Table 3.—*Relative traffic service of the primary, secondary, and third-class highway systems*

Highway system	Length of system	Portion of total highway mileage in each system	Daily traffic service by each system	Portion of total daily traffic service rendered by each system	Index of relative daily traffic service (third class=100 per cent)
	Miles	*Per cent*	*Vehicle miles*	*Per cent*	*Per cent*
Primary	1,630	7.1	1,702,000	53.4	340
Secondary	4,049	17.5	986,000	30.9	197
Third class	17,425	75.4	499,000	15.7	100

The importance of the primary system to motor-vehicle users is evident from the fact that 53.4 per cent of the total daily vehicle mileage on all highways in Maine is found on the primary system, which includes only 7.1 per cent of the total highway mileage. The slight traffic importance of the third-class system is evidenced by the fact that only 15.7 per cent of the total daily vehicle mileage is found on this system, which includes 75.4 per cent of the total highway mileage.

In these analyses of the relations between the three systems in Maine it must be borne in mind that the relations both as to density of traffic and daily vehicle mileage depend upon two elements which restrict their application more or less closely to the existing situation in Maine. These elements are the existing density of traffic on each mile of the Maine highways and the mileage of the three systems. Obviously, the expansion of the primary system by including 1,000 miles of highway now a part of the secondary system would materially change the relative daily vehicle mileage on the two systems. Similarly, a change in the mileage included in each system would also affect the average density of traffic on the three systems. The primary system, in general, now includes the more important highways of the State; the inclusion of a considerable mileage of less important highways would result in lowering the average density of traffic on the system.

It is evident, therefore, that the relationships between the three systems are true only for these systems as they exist to-day, and that any change in the systems will modify the relationships; and they are applicable in other States only in so far as the factors producing highway traffic and the proportion of highway mileage in the several systems of such States is comparable with the existing conditions in Maine.

Moreover, these relationships apply only to the three systems in Maine considered as units. Analysis of the traffic on the roads included in the sytems shows that there are material differences in the traffic importance of roads within each system.

COMPARISON OF TRAFFIC ON THREE SECTIONS OF THE PRIMARY SYSTEM

In Table 4 traffic is analyzed on three sections of the primary system. Section 1 includes route 1, from Kittery to Belfast; route 20, from Brunswick to Fairfield; route 100, from Portland to Augusta; and route 196, from Brunswick to Auburn—a total of 300 miles. Section 2 includes route 1, Ellsworth to St. Stephan; route 15, Oldtown to Houlton; route 20, Fairfield to the north State line; and route 24, from Houlton to the north State line—a total of 467 miles. All other routes of the primary system are grouped under section 3.

Table 4.—*Highway mileage and traffic on three sections of the primary system*

Section number	Length of section	Portion of primary system mileage in section	Average density of traffic	Daily motor-vehicle mileage on each system	Portion of total daily vehicle mileage on each system
	Miles	*Per cent*	*Vehicles per day*	*Vehicle-miles*	*Per cent*
1	300	18.4	2,197	659,000	38.7
2	467	28.7	525	245,000	14.4
3	863	52.9	924	798,000	46.9
Total	1,630	100.0	1,044	1,702,000	100.0

From this analysis it will be seen that the average density of traffic on 300 miles of the primary system (section 1) is over four times as great as the average density on section 2, which includes 467 miles and over twice as great as the average density on the 863 miles which compose section 3. The entire primary system, which includes 7.1 per cent of the total highway mileage, serves 53.4 per cent of the total daily traffic in vehicle-miles. But 300 miles, or 18.4 per cent of the primary system, serves 38.7 per cent of the total traffic served by the 1,630 miles of the primary system. With respect to the entire highway system of the State these 300 miles constitute only 1.3 per cent of the total highway mileage, but they serve 20.7 per cent of the total traffic.

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[4] The daily vehicle mileage on any system is the product of the total number of vehicles operated over any part of the system during the day and the average trip mileage of those vehicles. But the total number of vehicles operated over any part of the system during the day is the sum of the densities observed at the stations on the system divided by the number of times each vehicle is counted. With any given number of stations the number of times each vehicle is counted is equal to the average trip mileage divided by the average distance between stations; e. g., if the average trip is 35 miles and the average distance between counting points is 1 mile, each vehicle will be counted on the average 35 times.

The mathematical derivation of the approximate method of computing vehicle mileage is as follows:

$$
\begin{aligned}
\text{Daily vehicle mileage} &= \frac{(\text{Sum of densities})\ (\text{Average trip mileage})}{\text{Number of times each vehicle is counted}} \\
&= \frac{(\text{Sum of densities})\ (\text{Average trip mileage})}{\dfrac{\text{Average trip mileage}}{\text{Average distance between stations}}} \\
&= (\text{Sum of densities})\ (\text{Average distance between stations}) \\
&= (\text{Sum of densities})\ \frac{\text{Highway mileage}}{\text{Number of stations}} \\
&= \frac{\text{Sum of densities}}{\text{Number of stations}}\ (\text{Highway mileage}) \\
&= (\text{Average density})\ (\text{Highway mileage})
\end{aligned}
$$

The importance of section 1 is further illustrated by the comparison of the average daily gross tonnage per mile moved over each of the three sections of the primary system, which is shown in Table 5.

TABLE 5.—*Average daily gross tonnage per mile moved over three sections of the primary system*

Section No.	Average daily gross tonnage per mile		
	Passenger cars	Motor trucks	Total
	Tons	*Tons*	*Tons*
1	2,859	305	3,164
2	662	101	763
3	1,159	200	1,359

This table shows that in point of tonnage per mile, as well as density of traffic, section 1 of the primary system is over four times as important as section 2 and over twice as important as section 3.

Still another indication of the importance of the principal roads of the primary system is presented by Table 6, which shows the average maximum density of traffic on certain roads, as observed on Sundays during the period of the traffic survey, July 1 to October 31, 1924.

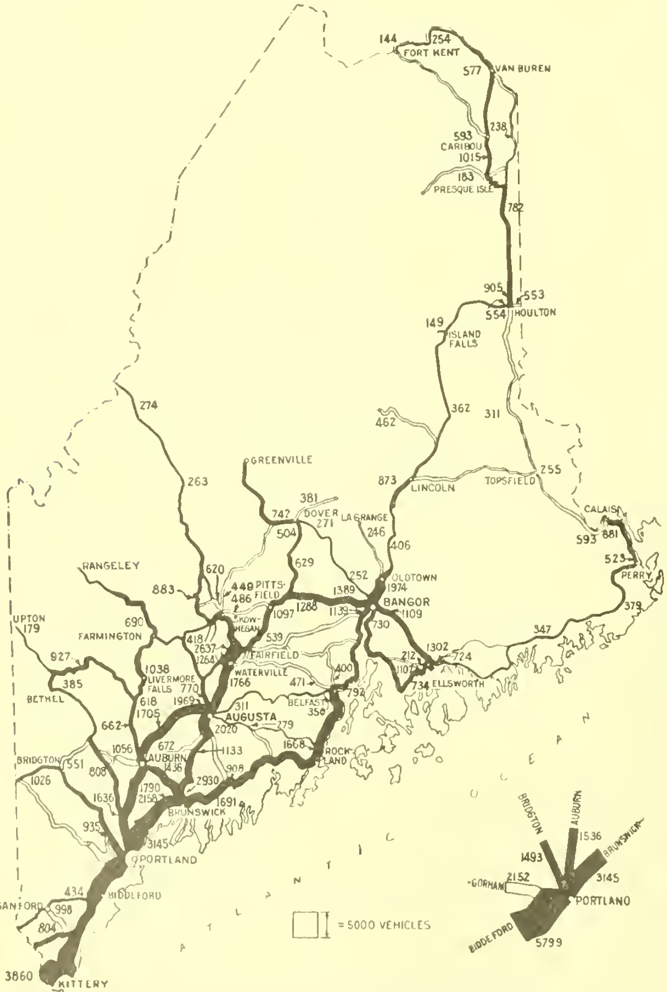

FIG. 1.—Average density of all motor-vehicle traffic on principal primary and secondary highways in Maine, July 1 to October 31, 1924

TABLE 6.—*Average maximum density of traffic on certain roads o the primary system (observed on Sundays during the period Jul 1 to October 31, 1924)*

Route and location of station	Average maximum density of traffic (Sunday)		
	Motor trucks	Passenger cars	Total
	Vehicles per day	*Vehicles per day*	*Vehicles per day*
Route 1:			
South of Portland	213	9,781	9,99
State line (south)	68	6,780	6,84
North of Portland	98	5,367	5,46
West of Brunswick	70	3,673	3,74
Southwest of Rockland	87	2,695	2,78
Route 20, north of Winslow	122	4,357	4,47
Route 100, west of Augusta	82	3,285	3,36
Route 196:			
Southeast of Auburn	77	2,335	2,41
North of Brunswick	55	5,132	5,18

A recapitulation of all data indicating the utilization of the highways of the three systems is given in Table 7.

TABLE 7.—*Motor-vehicle utilization of Maine highways, July 1 to October 31, 1924*

	All highways	Primary system	Secondary system	Third-clas system
Highway mileage	23,104	1,630	4,049	17,42
Percentage of highway mileage	100.0	7.1	17.5	75.
Daily vehicle-miles:				
All vehicles	3,187,000	1,702,000	986,000	499,00
Passenger cars	2,904,000	1,548,000	893,000	463,00
Trucks	283,000	154,000	93,000	36,00
Average density of traffic:[1]				
All vehicles	138	1,044	244	2
Passenger cars	126	950	221	2
Trucks	12	94	23	
Total vehicle-miles, July 1 to Oct. 31, 1924:				
All vehicles	392,001,000	209,346,000	121,278,000	61,377,00
Passenger cars	357,192,000	190,404,000	109,839,000	56,949,00
Trucks	34,809,000	18,942,000	11,439,000	4,428,00
Percentage of vehicle miles:				
All vehicles	100.0	53.4	30.9	15.
Passenger cars	100.0	53.3	30.8	15.
Trucks	100.0	54.4	32.9	12.
Average daily gross tons per mile:				
All vehicles	201	1,521	356	4
Passenger cars	176	1,330	309	3
Trucks	25	191	47	

[1] The average density of traffic is the weighted average density per day reduced t the nearest whole number. These average values were obtained by weighting th average daily number of vehicles at each station, or group of similar stations, by th number of miles of highway on which the daily traffic was approximately equal t this average, and therefore approximates the exact average obtained by summing th vehicles per day on each mile of highway and dividing the total by the number o miles of highway.

RELATIVE TRAFFIC IMPORTANCE OF PRINCIPAL ROADS

The relative traffic importance of the principal roads of the Maine highway system, as determined by the average density of traffic, is shown clearly in Figure 1 in which the density of traffic on roads of the primary and secondary systems is represented by the width of the lines. The chart shows at a glance which roads require the largest outlay of construction funds to provide adequate highway service.

The greatest density of traffic is found near the centers of population—Portland, Auburn, Lewiston Augusta, Brunswick, Waterville, Bangor—and the summer-resort district. Clearly the 300 miles previously defined as section 1 of the primary system constitute the backbone of the entire system. These roads from Kittery to Belfast, from Brunswick to Fairfield from Portland to Augusta, via Auburn, and from Brunswick to Auburn have, markedly, a greater density of traffic than any others in the State. Excluding these and the roads from Fairfield to Bangor and from Bangor to Ellsworth and Oldtown, the average density of traffic on the balance of the Maine highway system will not exceed 1,000 vehicles per day.

On a considerable mileage of the primary system and a large mileage of the secondary system the average density of traffic was less than 300 vehicles per day during the four months of the survey. It is anticipated that the density on these routes will not exceed 600 vehicles per day by 1930. No large expenditures will be required for high-type improvements on these roads for some years. They should receive only a sufficient amount of the construction and maintenance funds each year to meet their actual traffic needs, and the major portion of such funds should be devoted to the immediate improvement of the heavy-traffic roads.

Clearly, there is need for the creation of traffic zones bringing together for construction and maintenance purposes those sections of the highway system which serve approximately the same amount and type of traffic and to distinguish between those routes which require constant supervision and policing to insure satisfactory service and safety to traffic and those which do not.

In the main it is evident that the primary system includes the principal traffic arteries of the State and is therefore well selected. On the basis of their traffic density, however, some roads now included in the secondary system could more properly be included in the primary system than some that are included. The roads which could well be transferred to the primary system are those from Wells to Sanford, from Portland to Standish, from Auburn to Mechanic Falls, and from Oakland to Norridgewock.

The density of motor-truck traffic on the principal roads of the primary and secondary systems is shown in Figure 2. Obviously, the principal motor-truck routes in the State are those from Kittery to Portland, Portland to Augusta via Auburn, Portland to Brunswick, Lisbon Falls to Auburn, Auburn to Mechanic Falls, Waterville to Fairfield, Thomaston to Camden, Bangor to Ellsworth, Bangor to Oldtown, and Portland to Naples.

In general it will be recognized that these routes which have the largest daily motor-truck traffic are, with few exceptions, identical with those which have the greatest density of total traffic. The link of the primary system from Houlton to Van Buren via Easton and Presque Isle shows a relatively higher proportion of trucks to total traffic than is to be found on the primary system as a whole. So, also, does the route from Bangor to Ellsworth, a highway urgently in need of improvement. The motor-truck capacity analysis of the road from Portland to Naples, which also shows a heavy proportion of trucks to total traffic, indicates that this road carries an unusually large proportion of the heavier trucks.

The general use of motor trucks on the principal traffic routes indicates the need for highway improvement of a type adequate to provide service for truck traffic. The fact that the highway between Brunswick and Waldoboro is surfaced with gravel may be, in part, responsible for the small density of motor-truck traffic on this route. But the chart of motor-truck traffic density confirms the conclusion drawn from the discussion of the density of total traffic that high-type improvement of a considerable mileage of the primary and a large mileage of the secondary system can safely be deferred for some years.

Sections of the secondary system which have a considerable density of truck traffic are the roads from Portland to Standish and from Auburn to Mechanic Falls. These roads were shown to be eligible for transfer to the primary system by the analysis of total traffic.

STATUS OF ROAD IMPROVEMENT IN RELATION TO TRAFFIC DENSITY

The free flow of traffic on the principal sections of the primary system is hindered by gaps of unimproved highways or sections surfaced with gravel. Between Kittery and Portland, as shown in Figures 3 and 4, there are sections of concrete, macadam, and gravel. Sections of gravel road on a heavy-traffic route such as this do not provide adequate service to traffic and the yearly maintenance costs are excessive.

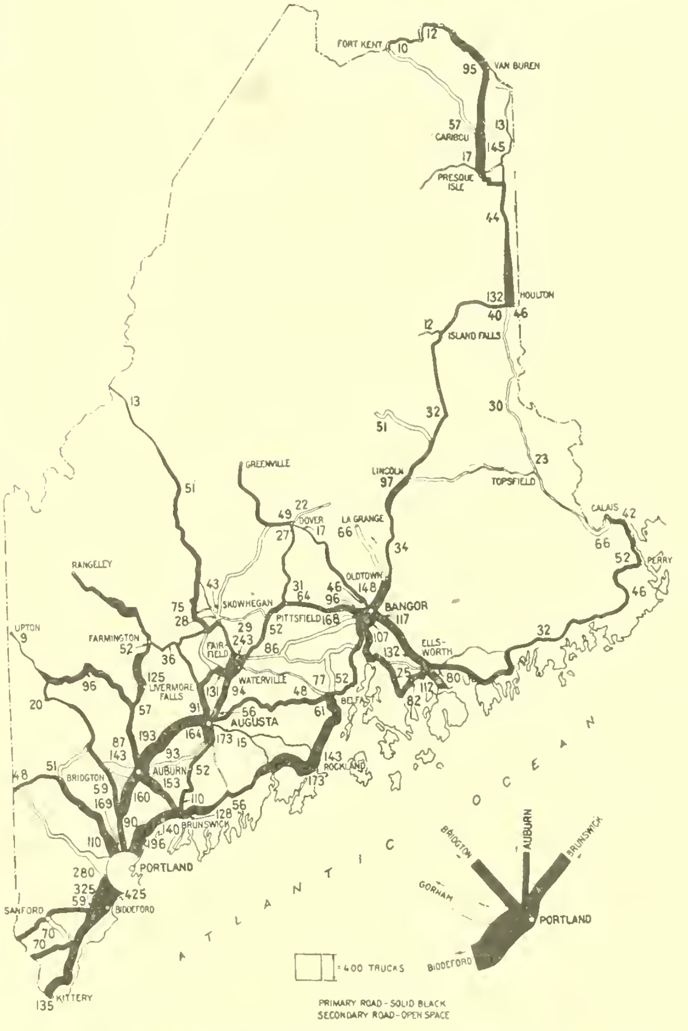

FIG. 2.—Average density of truck traffic on principal primary and secondary highways in Maine, July 1 to October 31, 1924

Between Auburn and Augusta, on one of the most heavily traveled routes in the State, approximately 16 of the 31 miles are gravel. Between Brunswick and Belfast there is a large mileage of gravel that undoubtedly does not give satisfactory service to the dense traffic. Between Brunswick and Augusta, also, there are sections of macadam, gravel, and unimproved road. After improvement the density of traffic over this route can be expected to increase materially.

The routes from Bangor to Ellsworth, one direct and the other via Orland, illustrate the need of improvement in proportion to the density of traffic. The direct route is gravel, although it is an important trucking route and is also of considerable importance as a passenger-car route. On the Bangor-Orland-Ellsworth route there is large unimproved mileage. The passenger-car movement and the surprisingly large motor-truck traffic over a large part of the route justifies at least a gravel surface.

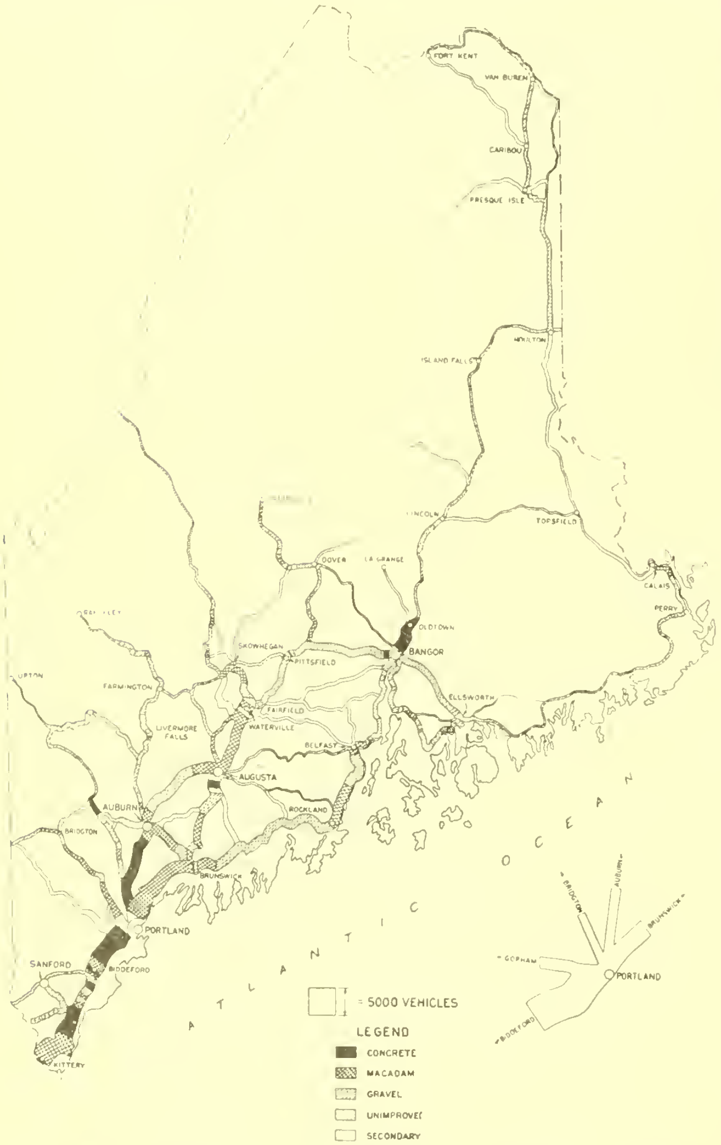

Fig. 3. Average density of motor vehicle traffic and road types on the primary system of Maine

In the northern part of the State the heavy trucking is caused largely by the transportation of potatoes. The average density of motor-truck traffic in this part of the State exceeds that on a large mileage of the primary system in the southeastern part of the State and apparently justifies a higher type of highway.

MOTOR-BUS AND HORSE-DRAWN TRAFFIC

Motor-bus traffic was recorded at very few stations, and even on the routes on which busses were operated their number was negligible.

The daily variation in the number of horse-drawn conveyances was found to be much greater than the variation in motor vehicles. At different stations the proportion of horse-drawn traffic to total traffic varies from less than 1 to 20 per cent. In general the greatest density of horse-drawn traffic was recorded at the stations at which the motor-vehicle traffic was light, although heavy horse-drawn traffic was found at some heavy motor-vehicle traffic stations which were located near towns. Extremely light horse-drawn traffic was found at some of the outlying stations which are located in undeveloped sections of the State.

The heaviest proportion of horse-drawn traffic was observed at stations located in well-developed agricultural areas which are off the main lines of motor-vehicle travel and which therefore have little motor-vehicle traffic. The horse-drawn traffic during September and October was considerably higher at the majority of stations than it was during the months of July and August. This increase was undoubtedly due to the crop marketing movement during the early fall months.

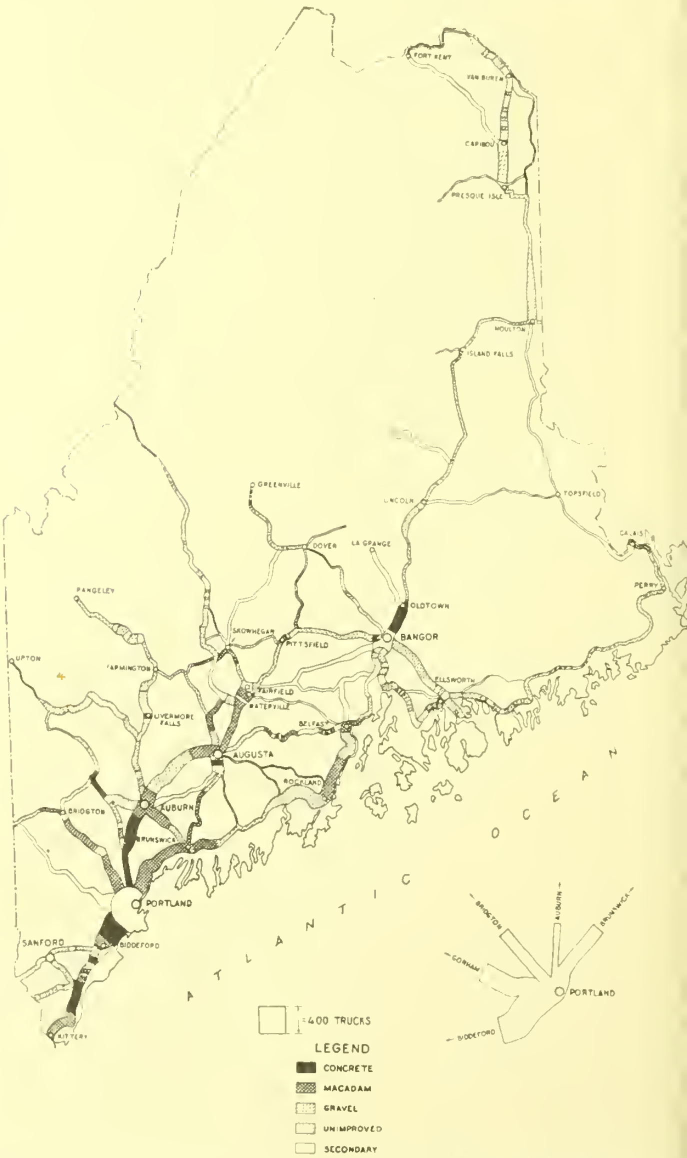

Fig. 4.—Average density of motor-truck traffic and road types on the primary system Maine

ANALYSIS OF MOTOR-TRUCK CAPACITY AND WHEEL LOADS

Motor trucks in use on the Maine highway system are predominantly of the smaller capacities, from one-half to 1½ tons. Even on the heavily traveled sections of the primary system practically all the trucks fall within the capacity group of one-half to 2½ tons, as shown by Table 8.

Table 8.—*Motor-truck capacities on the primary system*

Capacity group (tons)	Section 1		Section 2	
	Number	Per cent	Number	Per cent
½ to 2½	6,681	96.7	1,064	98.1
3 to 4	207	3.0	15	1.4
5 and over	23	0.3	5	0.5

Table 9 presents a summary, in greater detail, of the percentages of trucks of the various capacities observed at each weight station.

Table 9.—*Summary of the distribution of loaded trucks by capacity groups at weight stations*

Station	Distribution by capacity groups				
	½ to 1½ tons	2 to 2½ tons	3 to 4 tons	5 to 5½ tons	6 to 7½ tons
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*
401	87.1	5.5	7.4		
402	89.7	7.6	2.7		
403	93.9	5.9	.2		
404	86.8	10.3	2.9		
405	95.0	3.7	1.1	0.2	
406	84.3	12.3	2.8	.6	
407	78.3	13.0	7.2	1.5	
408	81.6	12.7	4.9	.8	
409	86.6	10.0	3.1	.3	
410	89.3	9.4	1.3		
421	82.3	13.1	3.0	.5	1.1
422	92.1	1.4	5.8	.7	
423	93.1	6.4	.5		
424	86.6	10.8	2.5	.1	
425	100.0				
426	87.5	12.5			
427	85.9	10.0	4.1		
428	81.5	10.9	7.6		
429	92.4	4.8	2.8		
430	91.5	4.0	4.5		

At every weight station but one (station 407) more than 80 per cent of the truck traffic consists of trucks of less than 2 tons capacity. At 7 of the 20 stations over 90 per cent of the traffic is composed of trucks of less than 2 tons capacity. Five-ton trucks are found at only 8 of the 20 stations, and the highest proportion of 5-ton trucks at any station is 1.5 per cent (station 407). Trucks larger than the 5 to 5½ ton group are found at only one station (station 421), and this is undoubtedly an exceptional movement. At all stations over 90 per cent of the truck traffic is made up of trucks of less than 3 tons capacity. Station 406, although having a smaller percentage of large trucks than some of the other stations, actually has a larger number of heavy trucks per day than does any other station. For use in the control of design of pavement to meet traffic requirements these percentages should be applied to the average daily truck density.

Except at stations 407 and 408 over 90 per cent of the loaded trucks at all weight stations were under 12,000 pounds gross weight. Trucks weighing over 24,000 pounds gross were found at only two stations (stations 407 and 408), and the percentages of such trucks at these stations was very small, 0.3 per cent at station 407 and 0.1 per cent at station 408. Trucks weighing between 12,000 pounds and 18,000 pounds gross were found at 11 of the 20 stations, but the highest percentage of such trucks at any station was 3.4 per cent (station 407). The percentage of loaded trucks weighing less than 12,000 pounds gross agrees quite closely with the percentage of trucks of less than 3 tons capacity, as shown in Table 10.

Table 10.—*Comparison of the percentages of trucks under 3 tons capacity and under 12,000 pounds gross load by stations*

Station	Trucks under 3 tons capacity	Trucks under 12,000 pounds gross weight	Station	Trucks under 3 tons capacity	Trucks under 12,000 pounds gross weight
	Per cent	*Per cent*		*Per cent*	*Per cent*
401	92.6	93.6	421	95.4	96.5
402	97.3	95.5	422	93.5	95.0
403	99.8	98.3	423	99.5	99.5
404	97.1	96.0	424	97.4	97.2
405	98.7	98.9	425	100.0	100.0
406	96.6	95.2	426	100.0	99.6
407	91.3	87.3	427	95.9	95.8
408	94.3	89.9	428	92.4	100.0
409	96.6	95.5	429	97.2	99.5
410	98.7	95.3	430	95.5	100.0

A comparison of the percentages in Table 10 indicates that for roads in Maine where truck traffic consists exclusively of trucks of less than 3 tons capacity it is fairly safe to select a type of pavement which will carry a maximum load of 12,000 pounds.

Over 80 per cent of the trucks on Maine highways are equipped with pneumatic tires. Practically all trucks weighing less than 6,000 pounds gross are so equipped, and although the percentage of pneumatic-tired trucks weighing between 6,000 and 12,000 pounds gross varies considerably with location it will average approximately 75 per cent of the total.

The observations indicate that the maximum wheel load of trucks whose gross load is under 6,000 pounds is 3,000 pounds, that wheel loads over 2,500 pounds are very exceptional, and that 95.9 per cent of the trucks under 6,000 pounds gross weight have rear-wheel loads of less than 2,000 pounds. For gross loads of less than 12,000 pounds 5,000-pound wheel loads are the maximum, and of all gross loads less than 12,000 pounds 98.5 per cent have wheel loads of less than 4,500 pounds.

PROPORTION OF LOADED TRUCKS OVER 50 PER CENT

The ratio of loaded to total trucks varies from 54.8 to 66.6 per cent. The lowest percentage is found at station 425 and the highest at station 401. Of the four stations at which less than 60 per cent of the trucks observed were loaded three are located on outlying routes—station 425 near Columbia Falls, station 423 south of Mattawamkeag, and station 402 east of Rumford. The fourth, station 403, is on route 100 between Auburn and Augusta.

The average net weight per loaded truck varies from 760 pounds at station 405 to 2,060 pounds at station 407, and the average gross weight from 3,420 pounds at station 425 to 6,180 pounds at station 407. Only two stations, 407 and 408, have an average net weight in excess of 1,600 pounds per loaded truck. These are also the only stations that show an average gross weight in excess of 5,000 pounds per loaded truck. Several of the stations of relatively heavy traffic density show very low average net and gross weights per loaded truck because of the preponderance of one-half-ton trucks. This is especially true of station 406, which has a larger number of 3 to 4 ton trucks and also a larger number of 5-ton trucks per day than stations 407 or 408, but because of the large number of small trucks the average weights are low. The average trip mileage per vehicle depends very largely upon the location of the station. Stations 407, 410, 421, and 425 show average trip mileages in excess of 35 miles; stations 402, 403, and 429 show average trip mileages below 20 miles.

USE OF MAINE HIGHWAYS BY FOREIGN MOTOR VEHICLES

When it is considered that 21.4 per cent of all motor vehicles on the primary system are of foreign registration, that the foreign vehicles account for an average density of traffic on the primary system amounting to 223 vehicles a day, and that these vehicles travel each day 364,000 vehicle-miles, it becomes evident that the cost of providing and maintaining adequate highways in Maine is increased by the usage of the roads by foreign vehicles.

Foreign passenger cars constitute 21.2 per cent of all motor vehicles on the primary system. They produce 23.3 per cent of the total passenger car-miles on the primary system, 9.9 per cent on the secondary system, and 6.5 per cent on the third-class system.

Foreign motor trucks are of much less importance. They account for only 2.1 per cent of the motor-truck-miles on the primary system, 1.6 per cent on the secondary system, and on the third-class system their influence is negligible.

Detailed data on the use of the Maine highways by foreign vehicles are given in Tables 11, 12, and 13.

Foreign passenger cars form a very important part of the total passenger-car traffic at stations near the State line and also at points a considerable distance from the State line on the principal traffic routes. As distance from the State line increases the proportion of foreign passenger cars decreases. On route 1 near Kittery (station 407), 68.2 per cent of the passenger cars carry foreign licenses. Near Wells (station 1B) the percentage is 43.5, and south of Portland the percentage is 34.5 (Table 14). North of Portland foreign passenger cars decrease to 29.2 per cent on route 1 (station 409) and 18.1 per cent on route 18 (station 408).

On route 1 the percentage of foreign passenger cars near Bath (station 405) is 27.6 per cent and near Rockland (station 429) 18.2 per cent. On route 20, south of Augusta (station 430), foreign passenger cars are 19.1 per cent of the total traffic.

Other routes in the State do not carry as large a proportion, but some foreign passenger cars were found at every station located on the primary and secondary system.

Trucks of foreign registration are of less importance. Foreign trucks make up over 10 per cent of the total truck traffic at only four stations, and these are all located near the State line. Only three stations (stations 406, 407 and 1B) show an average of more than 10 foreign trucks a day; only seven have an average of more than five foreign trucks a day; and a large number of stations have no foreign truck traffic. The difference in importance of foreign trucks and foreign passenger cars is indicated in Table 14, which presents data for three stations on route 1, south of Portland.

The comparative use of the Maine highway system by Maine and foreign vehicles is further evidenced by the average mileage of Maine and foreign passenger cars. Maine passenger cars average 36 miles per vehicle per trip on the primary system and 26 miles per vehicle per trip on the secondary system. Foreign passenger cars average 97 miles per vehicle per trip on the primary system and 73 miles per vehicle per trip on the secondary system.

TABLE 11.—*Utilization of Maine highway systems by all Maine and foreign motor vehicles*

System	Registration	Percentage of State and foreign vehicles	Average density of traffic, State and foreign vehicles	Daily vehicle-miles by State and foreign vehicles	Total vehicle-miles by State and foreign vehicles, July 1 to Oct. 31, 1924
		Per cent	*Vehicles par day*	*Vehicle-miles*	*Vehicle-miles*
Primary	Maine	78.6	821	1,338,000	164,546,000
Do	Foreign	21.4	223	364,000	44,800,000
Total		100.0	1,044	1,702,000	209,346,000
Secondary	Maine	91.0	222	897,000	110,363,000
Do	Foreign	9.0	22	89,000	10,915,000
Total		100.0	244	986,000	121,278,000
Third class	Maine	94.3	27	471,000	57,879,000
Do	Foreign	5.7	2	28,000	3,498,000
Total		100.0	29	499,000	61,377,000

TABLE 12.—*Utilization of Maine highway systems by Maine and foreign passenger cars*

System	Registration	Percentage of State and foreign vehicles	Average density of traffic, State and foreign vehicles	Daily vehicle-miles by State and foreign vehicles	Total vehicle-miles by State and foreign vehicles, July 1 to Oct. 31, 1924
		Per cent	*Vehicles per day*	*Vehicle-miles*	*Vehicle-miles*
Primary	Maine	76.7	729	1,187,000	146,040,000
Do	Foreign	23.3	221	361,000	44,364,000
Total		100.0	950	1,548,000	190,404,000
Secondary	Maine	90.1	199	805,000	98,965,000
Do	Foreign	9.9	22	88,000	10,874,000
Total		100.0	221	893,000	109,839,000
Third class	Maine	93.5	25	433,000	53,247,000
Do	Foreign	6.5	2	30,000	3,702,000
Total		100.0	27	463,000	56,949,000

TABLE 13.—*Utilization of Maine highway systems by Maine and foreign motor trucks*

System	Registration	Percentage of State and foreign vehicles	Average density of traffic, State and foreign vehicles	Daily vehicle-miles by State and foreign vehicles	Total vehicle-miles by State and foreign vehicles, July 1 to Oct. 31, 1924
		Per cent	*Vehicles per day*	*Vehicle-miles*	*Vehicle-miles*
Primary	Maine	97.9	92	150,800	18,544,000
Do	Foreign	2.1	2	3,200	398,000
Total		100.0	94	154,000	18,942,000
Secondary	Maine	98.4	22.6	91,500	11,256,000
Do	Foreign	1.6	.4	1,500	183,000
Total		100.0	23.0	93,000	11,439,000
Third class	Maine	100.0	2	36,000	4,428,000
Do	Foreign	----	----	----	----
Total		100.0	2	36,000	4,428,000

Table 14.—*Foreign passenger cars and trucks on route 1, south of Portland*

Station	Foreign passenger cars		Foreign trucks		Location
	Percentage	Average daily number	Percentage	Average daily number	
407	68.2	2,540	12.8	17.3	Near State line at Kittery.
1B	43.5	1,197	5.5	17.9	Near Wells, half way from State line to Portland.
406	34.5	1,854	2.7	11.5	Near Portland.

GASOLINE-TAX REVENUES

All motor vehicles using the Maine highways contribute toward the upkeep of these highways through the payment of a gasoline tax of 1 cent per gallon on all gasoline purchased in the State.

The total receipts from the gasoline tax during the period July 1 to October 31, 1924, were approximately $286,400. Assuming uniform consumption of gasoline per vehicle-mile and basing the calculation on the percentage of vehicle mileage as observed, the amount of revenue derived through the gasoline tax from each system would be $152,900 from the primary system, $88,500 from the secondary system, and $45,500 from the third-class system.

The total receipts from the gasoline tax for the calendar year 1924 were approximately $522,000. Assuming that the percentage of vehicle-miles on the three highway systems remains throughout the year the same as during the period of the survey (July 1 to October 31), the annual gasoline-tax revenues derived from each system are shown in Table 15, which also shows the revenue per mile on each of the three systems, as well as the revenue that could be collected from the same traffic at tax rates of 2 and 3 cents per gallon.

Table 15.—*Annual gasoline-tax revenues by highway systems, based on 1924 revenues and traffic from July 1 to October 31, 1924*

Highway system	Mileage	Tax of 1 cent per gallon		Tax of 2 cents per gallon		Tax of 3 cents per gallon	
		Total revenue	Revenue per mile	Total revenue	Revenue per mile	Total revenue	Revenue per mile
All highways	23,104	$522,000	$22.59	$1,044,000	$45.19	$1,566,000	$67.78
Primary	1,630	279,000	171.17	558,000	342.33	837,000	513.50
Secondary	4,049	161,000	39.76	322,000	79.53	483,000	119.29
Third class	17,425	82,000	4.71	164,000	9.41	246,000	14.12

The forecast of traffic for 1930 indicates that traffic will double during the period 1924 to 1930. Assuming that the average annual mileage per vehicle in 1930 remains the same as in 1924 and that no radical changes occur in the rate of gasoline consumption per vehicle-mile during this period, a gasoline tax of 1 cent per gallon in 1930 will produce $1,044,000, a tax of 2 cents per gallon $2,088,000, and a tax of 3 cents per gallon $3,132,000.

Foreign vehicles using Maine highways also contribute toward the upkeep of Maine highways by payment of the gasoline tax. In Table 11 the percentage of utilization by foreign vehicles is shown to be 21.4 per cent on the primary system, 9 per cent on the secondary system, and 5.7 per cent on the third-class system. Assuming that the foreign traffic during the month of June is equal to the foreign traffic during the average month of the period July 1 to October 31 and that during the remainder of the year it is insignificant, the gasoline tax receipts from foreign vehicles during 1924 were approximately: Primary system, $40,900; secondary system, $10,000; third-class system, $3,200.

The amount of the receipts which on the same basis would be derived in a year from foreign vehicles on the three highway systems at tax rates of 1, 2, and 3 cents per gallon of gasoline are shown in Table 16.

Table 16.—*Annual gasoline-tax receipts from foreign vehicles by highway systems, based on 1924 traffic*

Highway system	Tax of 1 cent per gallon		Tax of 2 cents per gallon		Tax of 3 cents per gallon	
	Total revenue	Revenue per mile	Total revenue	Revenue per mile	Total revenue	Revenue per mile
Primary	$40,900	$25.09	$81,800	$50.18	$122,700	$75.28
Secondary	10,000	2.47	20,000	4.94	30,000	7.41
Third class	3,200	.18	6,400	.37	9,600	.55

IMPROVEMENT OF PRINCIPAL ROADS FULLY JUSTIFIED BY TRAFFIC SAVINGS

Whether or not the State is economically justified in improving its roads and the degree of improvement warranted—these questions depend to a considerable extent upon the amount of the savings in the operating costs of vehicles made possible by the improvement. Assuming that the operating cost is reduced 2.7 cents per passenger-car-mile by constructing a concrete road where formerly there has been an earth road,[5] and applying this difference in operating costs to the average daily traffic on the heavy-traffic routes of the Maine primary system, the change in the type of construction is shown to be fully justified.

For example, the average density of passenger-car traffic on section 1 of the primary system, involving 300 miles of the most heavily traveled roads in the State, was 2,042 cars. The total number of passenger-car-miles per mile during the period from July 1 to October 31 was, then, approximately 251,000. As gasoline-tax receipts during this period were approximately 55 per cent of the total receipts for the year, it is reasonable to assume that the traffic during this period was approximately 55 per cent of the annual traffic. On this basis, the total passenger-car mileage per mile of the heavy-traffic routes of the primary system for the year 1924 would be approximately 456,000 passenger-car-miles; and the saving of 2.7 cents per passenger-car-mile effected by changing the type of the road from earth to concrete would be approximately $12,300 per mile. As the average cost of Federal-aid concrete roads completed in Maine up to July 30, 1924, was $45,200 per mile, it will be seen that the entire cost of building the concrete road could be retired by the saving in operating costs of $12,300 per mile in a few years. With interest at 4 per cent the time required would be slightly over four years.

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[5] The difference in operating cost of 2.7 cents per passenger-car-mile is based on the researches conducted by the Iowa engineering experiment station under the direction of T. R. Agg, results of which are published in Bulletin 69, Iowa State College of Agriculture and Mechanic Arts, by T. R. Agg and H. S. Carter. The costs are tentative and are applicable directly only to the surfaces and the vehicles used in the tests. They may not express the true relation of operating costs on concrete and earth roads in Maine, but they are the most reliable estimates available and seem reasonable.

It is believed that the 2.7 cents saving assumed above is a conservative estimate of the difference in operating costs on concrete and earth roads in Maine. But it will be noted that in the example cited the considerable operating savings of motor trucks, of which there were 155 a day on the heavy-traffic routes of the primary system, have been purposely ignored, as also have been savings resulting from the reduction in the cost of road maintenance. Indeed, it is highly probable that it would be impossible to maintain an earth road in satisfactory condition under the daily traffic on these roads which is in excess of 2,000 vehicles a day. Experience of the Maine Highway Commission with regard to maintenance costs on the more heavily traveled highways of the State indicates "that traffic is being carried on the bituminous macadam roads at a cost one-sixth as much per vehicle-mile as is the cost of carrying traffic on the gravel-surfaced highways."[6]

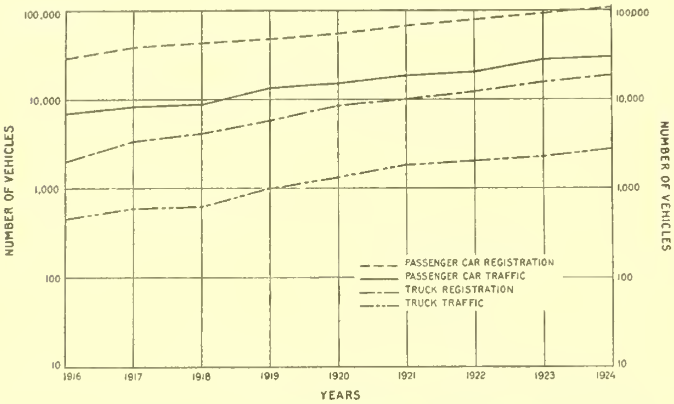

FIG. 5.—Maine highway traffic and motor vehicle registration, when plotted to a logarithmic scale, shows by the approximately equal slopes of the curves that the rates of increase of traffic and registration are nearly equal

Maintenance costs per vehicle-mile on a concrete highway probably would not be higher than on a bituminous macadam highway. Combining this saving in maintenance costs with the saving in vehicle operating costs, the economy of the higher-type surfaces on the more heavily traveled highways of the State becomes even more apparent.

FORECASTING OF TRAFFIC MADE POSSIBLE BY SURVEY

One of the most valuable results of a highway transportation survey is the development of fundamental traffic information as a basis for estimating, with reasonable accuracy, future highway traffic. A highway traffic forecast makes possible, for the period of the forecast at least, the development of a comprehensive highway program including the designation of routes to be improved, the order of their improvement, and the types of improvement required. The selection of the improvement type should be based not only upon a forecast of vehicle density but also upon the weight of traffic units obtained by a motor-truck capacity and gross weight analysis. The forecast stabilizes the highway program. Uncertainty as to the growth of traffic is largely eliminated and the highway department is able to project a definite plan of improvement over a period of years based on the growth of traffic. By establishing a definite plan of improvement highway development is carried on in a more efficient and economical manner, and addition economy is effected by the elimination of excessive maintenance costs on unsatisfactory types of highway surfaces.

It is possible by means of a forecast of traffic and registration to establish the highway budget for a reasonable period of years. Revenues to be derived from motor-vehicle license fees, gasoline taxation, and other sources of revenue can be predicted with reasonable accuracy. If such revenues, together with other available highway funds, are not sufficient to carry out the necessary program, the amount of bond issues (if the use of credit is justified in carrying out the program of improvement) can be determined. By estimating some time in advance the amount of bonds to be marketed in any one year (when the bond-issue method of raising revenue is necessary) it is possible to take advantage of the most economical market for the sale of these bonds.

GROWTH OF MAINE TRAFFIC, 1916 TO 1924

Fortunately, traffic figures are available in Maine for one week each year from 1916 to 1923, inclusive. During these years vehicles were counted from 7 a. m. to 7 p. m. for an entire week, either in the latter part of August or the beginning of September. Although the total number of stations in 1923 was 58, only 26 of these stations can be used for the whole period (1916–1923), and even in the case of these 26 stations it is necessary to interpolate some figures for the years 1916 and 1917. Beginning with 1918, however, traffic records are available for all of the 26 stations. The stations are widely distributed over the highway system, and the traffic density recorded each year is representative of the change in traffic on Maine highways from 1916 to 1923.

The average daily number of trucks and passenger cars has been computed from the week's record at each of the 26 stations, and these daily station averages have been combined to yield average figures representative of the traffic at the 26 stations in Table 17. These figures give a reliable indication of the growth of traffic over a period of years on the principal Maine highways.

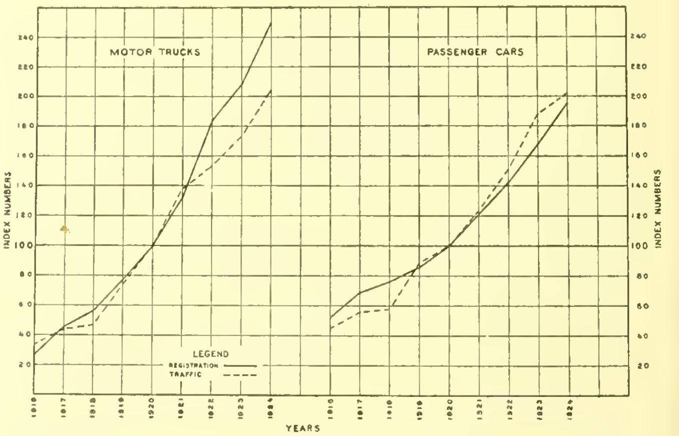

FIG. 6.—Over the period 1916 to 1924, motor- truck traffic on Maine highways increased at a slower rate than motor-truck registration; passenger-car traffic during the same period increased at a faster rate than passenger-car registration

Comparing with these figures the data shown in Table 18 representative of the registration of trucks and passenger cars in the State during the same period and plotting the data from the two tables to a logarithmic scale reveals the fact that the rates of increase of

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[6] Eleventh Annual Report of the Maine State Highway Commission, p. 17.

Table 17.—*Average daily traffic at 26 stations in Maine, 1916 to 1924*

Year	Trucks	Passenger cars	Total
1916	461	6,855	7,316
1917	584	8,392	8,976
1918	632	8,714	9,346
1919	989	13,479	14,468
1920	1,339	15,082	16,421
1921	1,848	18,484	20,232
1922	2,052	22,844	24,896
1923	2,317	28,252	30,569
1924 [1]	2,734	30,258	32,992

[1] Figures for 1924 estimated on the basis of those stations in the 1924 traffic census which were comparable with the above 26 stations.

traffic and registration are nearly equal, as shown by the approximately equal slopes of the registration and traffic curves in Figure 5. The same fact is brought out by Figures 6 and 7, in which the registration and traffic over the period are compared on the basis of index numbers derived by comparing the tabulated registration and traffic figures for each year, in one case, with the figures for 1920, and, in the other, with the average figures for the period, as a base. These charts should not be interpreted as meaning that the traffic and registration are equal throughout the series of years. They mean merely that the rates of increase of the two differ only slightly and that traffic and registration are in nearly constant ratio from year to year.

Since this is so, a forecast of registration will also predict future traffic, provided there is no important change in the average annual use per vehicle. As any change of this kind must, from the nature of the case, be gradual; such a change will not greatly affect the relation between traffic and registration during the next few years.

Table 18.—*Registration of motor vehicles in Maine, 1916 to 1924*

Year	Trucks	Passenger cars	Total
1916	1,991	28,991	30,972
1917	3,382	38,117	41,499
1918	4,200	42,372	46,572
1919	5,795	47,630	53,425
1920	7,512	55,395	62,907
1921	9,936	67,591	77,527
1922	13,842	78,697	92,539
1923	15,614	92,995	108,609
1924	18,779	107,933	126,712

Table 19 presents the data shown in Tables 17 and 18, as indices of the average year. The graphic presentation of these indices in Figure 7 shows clearly the close agreement in the rates of increase of traffic and registration.

Table 19.—*Relative growth of highway traffic and motor-vehicle registration (average of years 1916 to 1923=100)*

Year	Traffic	Registration
1916	44.2	48.3
1917	54.3	64.8
1918	56.5	69.6
1919	87.5	83.5
1920	99.3	98.3
1921	122.9	121.1
1922	150.5	144.6
1923	184.8	169.7
1924	199.5	198.0

FORECAST OF REGISTRATION AND TRAFFIC, 1924 TO 1930

This relation between the rates of increase of traffic and registration may be employed to predict traffic a reasonable number of years in advance by projection of the increase in vehicle registration. Future registration depends upon future population and car ownership per unit of population, and both of these factors can be projected on the basis of their trends over past years.

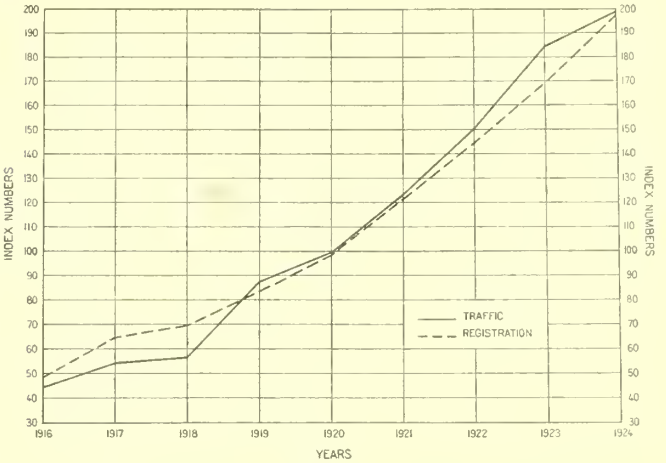

Fig. 7.—The parallel increase in the index numbers of traffic and registration indicates that, considering all vehicles, the rates of increase of traffic and registration in Maine from 1916 to 1924 were nearly equal

The registration of passenger cars and trucks per unit of population in Maine from 1916 to 1924, inclusive, is plotted as a solid line in Figures 8 and 9, respectively, and trend curves projected to 1930 from these actual figures by the method of least squares are shown by the dotted lines. The results of these projections and a comparison of the computed trend with the actual figures for the period 1916 to 1924 are shown in Table 20.

Table 20.—*Persons per motor vehicle, 1916 to 1930*

Year	Persons per passenger car		Persons per truck	
	Actual	Estimated	Actual	Estimated
1916	26.2	24.4	381	317
1917	20.2	21.7	225	257
1918	18.0	18.2	182	187
1919	16.1	15.4	132	137
1920	13.9	13.1	102	102
1921	11.4	11.3	77.7	78.1
1922	9.84	9.81	56.0	60.8
1923	8.36	8.62	49.8	48.1
1924	7.22	7.64	41.5	38.6
1925		6.82		31.4
1926		6.12		25.8
1927		5.53		21.4
1928		5.02		17.97
1929		4.57		15.18
1930		4.19		12.92

Applying the factors of estimated persons per passenger car and per truck (Table 20) to the expected population, the estimated registration is obtained as shown in Table 21.

The experienced and expected rates of increase in registration are shown in Table 22. The percentage increase for each of the years 1917 to 1924, inclusive, is computed from actual registrations shown in Table 18, and the percentage increase for each of the years 1925 to 1930, inclusive, is computed from predicted registration as shown in Table 21.

Table 21.—*Estimated motor-vehicle registration, 1925 to 1930*

Year	Population	Passenger-car registration		Motor-truck registration		Total registration	
		Number	Increase over 1924	Number	Increase over 1924	Number	Increase over 1924
			Per cent		*Per cent*		*Per cent*
1924	779, 902	[1] 107, 933	0. 0	[1] 18, 779	0. 0	[1] 126, 712	0. 0
1925	782, 544	114, 740	6. 3	24, 920	32. 6	139, 660	10. 1
1926	785, 186	128, 300	19. 0	30, 430	62. 2	158, 730	25. 2
1927	787, 828	142, 460	32. 0	36, 810	96. 0	179, 270	41. 5
1928	790, 470	157, 460	46. 0	43, 990	134. 2	201, 450	59. 0
1929	793, 112	173, 550	61. 0	52, 250	178. 1	225, 800	78. 0
1930	795, 754	189, 920	76. 0	61, 590	228. 0	251, 510	98. 5

[1] Actual 1924 registration.

From Table 21 it is apparent that motor-truck registration may be expected to increase at a faster rate than passenger-car registration. Figure 6 shows that motor truck traffic increases at a slower rate than truck registration and passenger-car traffic increases a little more rapidly than passenger-car registration; but for total vehicles the rates of increase of

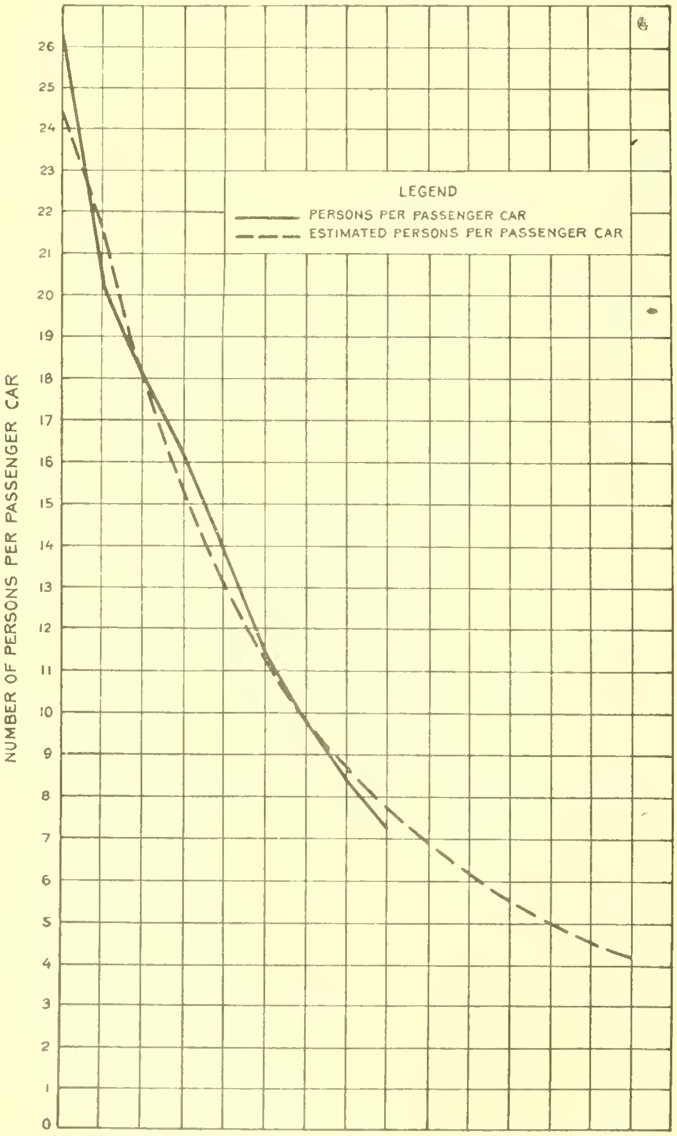

Fig. 8.—Persons per passenger car, 1916 to 1930

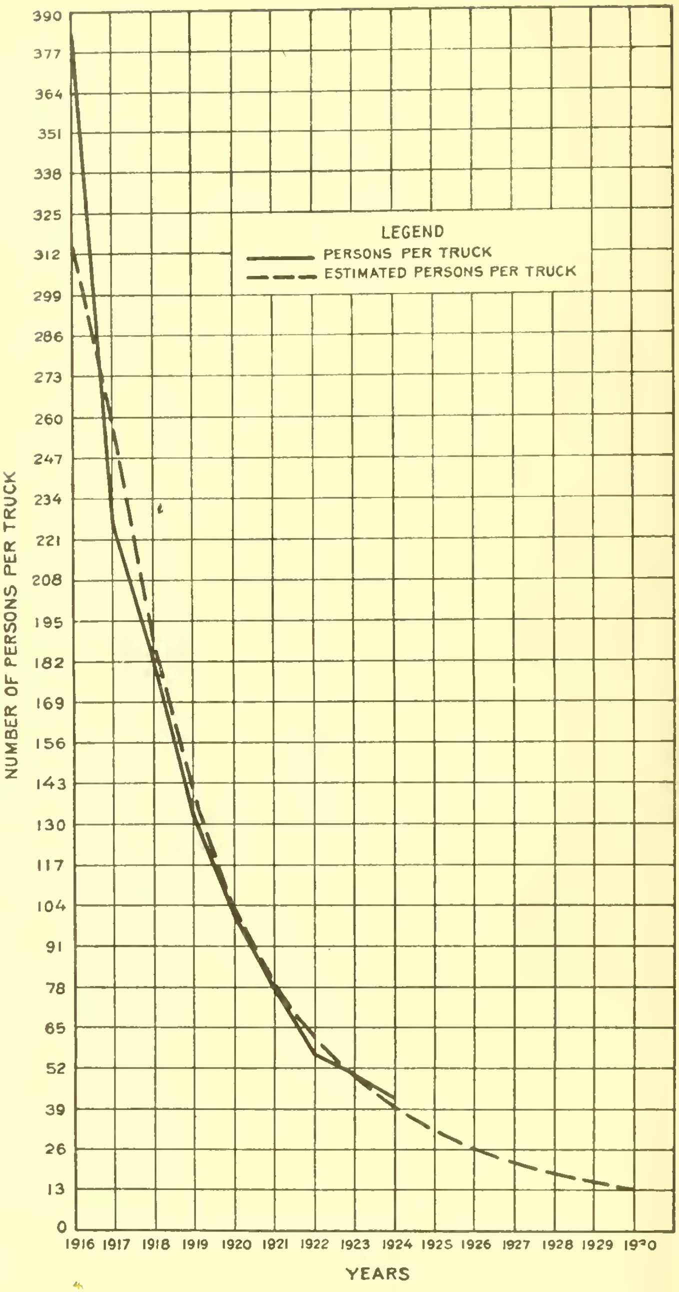

Fig. 9.—Persons per motor truck, 1916 to 1930

Table 22.—*Annual increase in total motor-vehicle registration, 1917 to 1930*

Year	Increase over preceding year	Year	Increase over preceding year
	Per cent		*Per cent*
1917	34. 0	1926	13. 7
1918	12. 2	1927	12. 9
1919	14. 7	1928	12. 4
1920	17. 7	1929	12. 1
1921	23. 2	1930	11. 4
1922	19. 4	Average annual increase 1917–1924	19. 4
1923	17. 4		
1924	16. 7	Average annual increase 1925–1930	12. 1
1925	10. 2		

traffic and registration have been nearly equal. The expected increase in total registration from 1924 to 1930 is 98.5 per cent, and, as shown by Figure 7, this rate of increase may be expected for the total traffic. Truck traffic will probably increase faster than passenger-car traffic but not in the ratio of the increase in registration as given in Table 21, because truck traffic on the Maine highway system has not increased as rapidly as truck registration in the State.

The expected increase in traffic of 98.5 per cent from 1924 to 1930 is applied to the 1924 traffic at all traffic stations of the 1924 survey. It is not expected that traffic at every station will increase at exactly this rate. Road improvements will modify the rate at certain stations. The opening of new routes or the development of new industries will affect the rate of increase; a new shore resort or a new route to an old resort would have considerable effect. Despite these known facts, however, a study of the traffic counts from 1916 to 1923 clearly indicates that the rate of increase at a majority of the stations varied but little from the rate of increase at all stations combined.

The expected traffic in 1930 at stations used in the 1924 traffic survey is given in Table 23. The figures are for trucks and passenger cars combined. It is anticipated that truck registration will increase at a faster rate than passenger-car registration, and the ratio of trucks to total vehicles on the road in 1930 will probably be a little greater than in 1924.

TABLE 23.—*Anticipated total traffic density in 1930 at all stations of the 1924 survey*

Station	Total anticipated traffic density, 1930	Station	Total anticipated traffic density, 1930	Station	Total anticipated traffic density, 1930
1A [1]	867	63	538	125	1,099
1B [2]	6,105	64	756	126A [1]	363
2	1,981	65	488	126B [2]	2,014
3	1,596	66	806	127	1,177
4	3,644	67	421	128	286
5	1,856	68	1,437	129	504
6	2,036	69	1,038	130	1,145
7	1,094	70	1,177	131	472
8	355	71	1,352	132	1,552
9	764	72	752	133	1,796
10	2,096	73	1,457	134	1,097
11	1,314	74	2,584	135	617
12	1,334	75	500	401	1,604
13	2,850	91	1,528	402	1,840
14	3,552	92	3,505	403	3,384
15	4,271	93	2,509	404	5,815
31	4,283	94	5,233	405	3,356
32	2,249	95	965	406	11,509
33	3,908	96	891	407	7,661
34	554	97	1,230	408	2,963
35	617	98	522	409	6,242
36	1,070	99	544	410	3,247
37	2,552	100	1,752	421	2,177
38	2,258	101	1,733	422	1,248
39	2,201	102	1,369	423	1,733
40	1,449	103	1,226	424	3,918
41	794	104	2,060	425	689
42	935	105	830	426	2,197
43	695	121	506	427	2,757
44	3,310	122	917	428	1,572
45	1,802	123	718	429	3,283
61	1,000	124	296	430	4,009
62	1,473				

[1] Before change in location; change occurred as follows: Station 1, Sept. 28, 1924; station 126, Aug. 28, 1924.
[2] After change in location.

TRAFFIC EXPECTED TO DOUBLE BETWEEN 1924 AND 1930

Summarizing the facts brought out in the foregoing discussion, it is found that Maine registration doubled in the four years from 1916 to 1920. It doubled again in the four years from 1920 to 1924, and the forecast anticipates that it will double again during the six years from 1924 to 1930, as shown in the following tabulation:

Maine motor-vehicle registration

Year	Total registration
1916	30,972
1920	62,907
1924	126,712
1930	[1] 251,510

[1] Estimated.

In almost exact coincidence Maine highway traffic doubled in the three years from 1916 to 1919; it doubled again during the four years from 1919 to 1923 (this is the more significant period, since the traffic figures for 1916 and 1917 are less reliable); and the forecast anticipates that it will double again during the six years from 1924 to 1930, as shown in the following tabulation:

Total daily traffic at 26 stations

Year.	Total daily traffic
1916	7,316
1919	14,468
1923	30,569
1930	[1] 60,527

[1] Estimated.

The Eleventh Annual Report of the Maine State Highway Commission gives a tabulation of the results of past traffic surveys, which differs somewhat in respect to the rate of increase from the figures presented above. The commission's figures are given in the first three columns of Table 24, in the fourth column of which are given, for comparative purposes, the indices of total traffic at the 26 stations included in the analysis reported herein. It will be noted that the number of stations averaged in the commission's analysis differed from year to year, while the analyses made in this report are based on the same 26 stations each year from 1916 to 1923, inclusive.

TABLE 24.—*Comparison of indices of traffic taken from report of Maine Highway Commission and this report*

Year	Number of stations	Average number of vehicles per day	Index (1920=100) [1]	Index of total traffic at 26 stations (1920=100) [2]
1916				45
1917	18	428	83	55
1918	19	483	94	57
1919	38	504	98	88
1920	41	515	100	100
1921	43	715	139	123
1922	46	767	149	152
1923	49	961	187	186

[1] Eleventh Annual Report, Maine State Highway Commission.
[2] Based on 26 stations, 1916–1923.

The commission's report states: "It is seen from the above figures that traffic in 1923 was two and one-fourth times as heavy on an average as in 1917. It should be borne in mind that new stations added from year to year were not located on the heaviest traveled highways. Several of these stations were off the State highway system, the object of the commission being to secure traffic data which might indicate the location of roads which eventually should be added to the State highway system."

Despite the difference in number of stations there is a close agreement between the indices of total traffic at the 26 stations and the indices at from 41 to 49 stations from 1920 to 1923. The number of stations from 1920 to 1923 is more nearly constant in the commission's report. But the indices of traffic based on data from a number of stations varying from 18 in 1917 to 49 in 1923 are misleading for prediction purposes. The adding or dropping of stations from year to year makes the average number of vehicles per day (per station) incomparable in the given years. The Maine report recognizes this defect in the note quoted above.

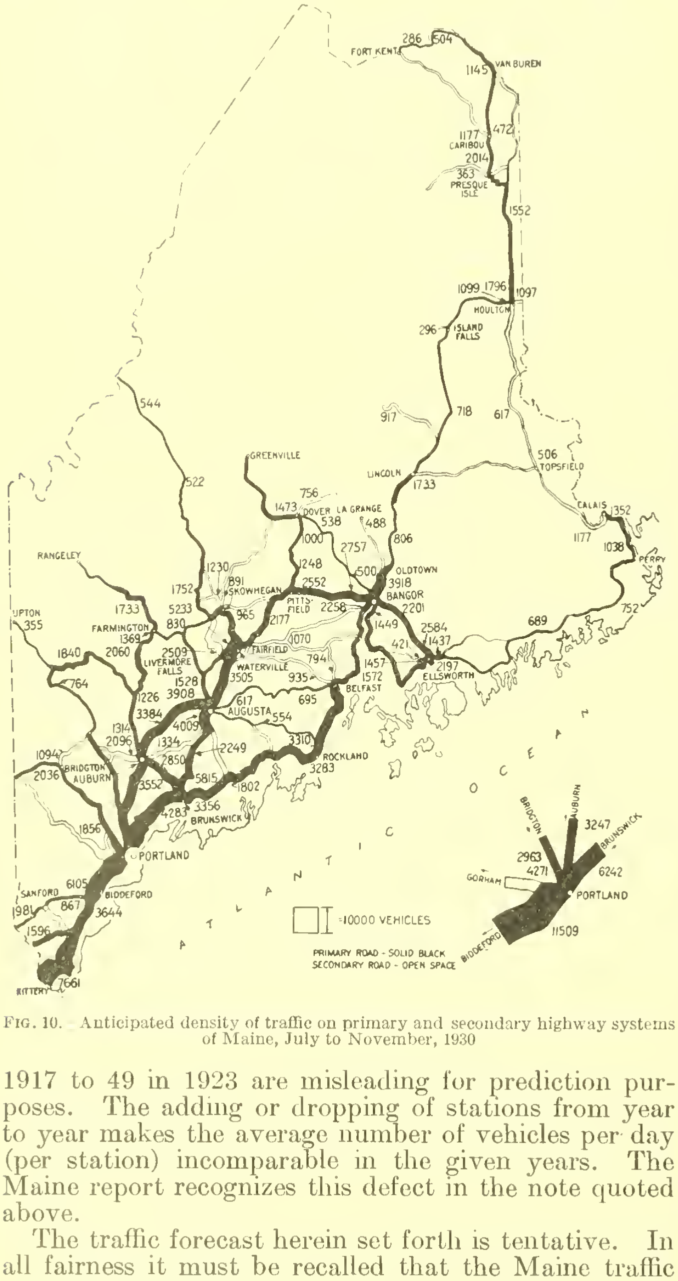

FIG. 10. Anticipated density of traffic on primary and secondary highway systems of Maine, July to November, 1930

The traffic forecast herein set forth is tentative. In all fairness it must be recalled that the Maine traffic records for the period 1916 to 1923 are for only one week per year, although that week is in the same part of each year. A second consideration, which is not so important in its effect upon the conclusions, is that data for 1916 and 1917 are missing at some of the stations, and therefore it has been necessary to interpolate some values for these years.

The location of the 26 stations used to measure the increase of traffic in Maine, 1916–1923, is a basis for confidence in the application of the increases there recorded to the 1924 traffic data, since the 26 stations are widely distributed throughout the State on the routes covered by the 1924 survey. Neglecting such factors as the effect of major mechanical improvements to automobiles, it is believed that the traffic estimates here given will closely represent traffic on Maine highways in 1930.

A definite betterment of the method of planning future highway improvements is represented by Figure 10, which shows a forecast of the average daily traffic on the principal primary and secondary routes during the period July to November, 1930. The problem of developing such a forecast in Maine was simplified by the fact that the Maine Highway Commission kept a record of traffic each year at a selected number of key stations from 1916 to 1924. As a number of State highway departments have had the foresight to preserve similar traffic records on their primary systems over a period of years, it should be possible to make similar forecasts of future traffic in these other States.

SUGGESTED PROGRAM OF HIGHWAY IMPROVEMENT

The forecast of expected registration and traffic is of particular importance at the present stage of highway development in Maine. The State has reached the second critical stage in its highway improvement program. The first stage may be called the gravel-road stage. During this first period traffic demanded highway service over a large mileage, and the highway commission wisely inaugurated the policy of stage construction based on the theory of providing highway service over a large mileage rather than a concentration of expenditures on a limited mileage. The gravel road met this demand. Traffic, however, was relatively light and, with the exception of a comparatively small mileage of heavy-traffic routes, gravel furnished a satisfactory and economical surface. The proper policy was to improve a large mileage of roads to the gravel stage and confine improvements of a higher type to a comparatively small mileage of heavy-traffic routes. The Maine Highway Commission has followed this policy in an excellent manner.

The second stage of a highway improvement program may properly be called the reconstruction and high-type improvement stage. When a State reaches this stage it is essential to set up a definite improvement policy for a period of years. Maine experience indicates that a gravel road will not carry over 500 vehicles per 12-hour day successfully unless it is surface treated.[7] The average daily density of traffic on the entire primary system for the period July 1 to October 31, 1924, was over 1,000 vehicles.

Clearly, Maine is now entering the reconstruction and high-type improvement stage. The questionable economy of attempting to maintain a gravel surface on heavy-traffic routes is demonstrated by the maintenance costs on sections of heavy-traffic gravel roads in the State.[8]

The 1923 maintenance costs per mile per vehicle on the Waterville–Bangor highway are more than double such costs during 1919 and 1920. Traffic has practically doubled during the same period. Similar maintenance costs per mile per vehicle for 1923 on the Woolwich–Bangor highway are double the costs for the years 1918 and 1919. Tables in the same report[9] indicate

(Continued on page 67)

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[7] Eleventh Annual Report, Maine State Highway Commission, p. 17

[8] Ibid., p. 15.

[9] Ibid., pp. 16–17

THE WAGON AND THE ELEVATING GRADER

AN ECONOMIC STUDY OF THE WAGON—ELEVATING GRADER COMBINATION IN THREE PARTS

BY THE DIVISION OF CONTROL U. S. BUREAU OF PUBLIC ROADS

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PART II.—THE INFLUENCE OF DESIGN ON ELEVATING GRADER COSTS

THE factors influencing the production of elevating grader-wagon outfits and the conditions which must obtain if a high rate of production is to be secured were discussed in Part I of this article. The proper adjustment of the size of the wagon train to the special conditions of the project in order, (1) that the number of wagons shall be sufficient to move the output of the grader as rapidly as it is produced, and (2) that their number shall not exceed the supply that can be utilized without periods of idleness by the grader, was shown to be dependent not only upon the output of the grader but also upon the distance the excavation must be moved and the rate of wagon travel. The length of haul is an especially important factor in elevating grader work, and it is peculiarly affected by the design of the road.

In all grading operations rational bidding must take account of the haul as a controlling factor. This is a comparatively simple matter when the material is moved with the simpler forms of equipment, such as the wheeler and the fresno. When these implements are used cuts can generally be taken out substantially as indicated by the balance points shown on ordinary highway plans, and the actual haul will therefore be found to be closely in agreement with the calculated haul, so that a knowledge of the average calculated haul generally suffices as a general basis for estimating.

But such data are inadequate for careful bidding on elevating grader work. In the first place the transfer of the loading function to an independent apparatus introduces an expensive operation which, whether the haul be long or short, accounts for a larger part of the total cost than the corresponding operation on wheeler and fresno jobs. In the second place the movement of the material in the large volumes made possible by the wagons reduces the cost of movement, so that it becomes proportionately less important than on wheeler and fresno jobs. By reason of both of these differences the length of haul on elevating grader jobs is less important in its bearing on costs than on the wheeler or the fresno job, and the average haul is relatively less significant.

But, though the length of haul is less significant than with other methods of operation, the fluctuation in haul distance, which is an inseparable feature of the work, does have a most important bearing on the cost because of its effect on the relation of the moving to the loading element, i. e., the wagons supply to the grader. It is customary for the contractor to supply a fixed number of wagons with the elevating grader. The supply furnished may be in proper relation to the grader output for the average haul; but from cut to cut, and even in the same cut, the haul will often vary between limits which, if the haul is short, leave the teams idle much of the time, and if the haul is long, leave the grader idle much of the time. For this reason the average haul is not a proper basis for estimating where elevating graders are to be used.

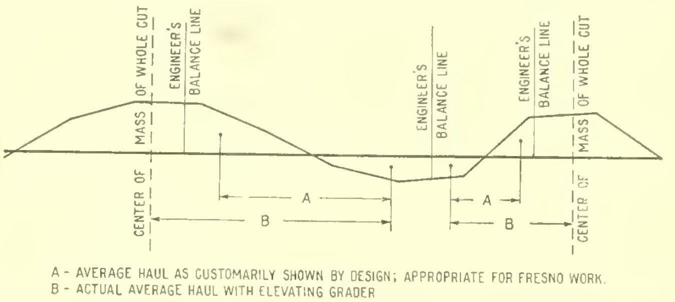
CENTER OF MASS OF WHOLE CUT
ENGINEER'S BALANCE LINE
ENGINEER'S BALANCE LINE
ENGINEER'S BALANCE LINE
CENTER OF MASS OF WHOLE CUT
A
B
A
B
A - AVERAGE HAUL AS CUSTOMARILY SHOWN BY DESIGN; APPROPRIATE FOR FRESNO WORK.
B - ACTUAL AVERAGE HAUL WITH ELEVATING GRADER

FIG. 1.—Diagram illustrating difference in average haul on fresno and elevating-grader jobs

PECULIARITIES OF HAUL ON ELEVATING GRADER JOBS

Moreover, the average haul on elevating grader work differs from the average haul as calculated for fresno or wheel-scraper work. With the latter form of equipment the length of haul grows as the cut and fill are extended, and the actual haul generally agrees closely with the haul distance as shown on the plans. But the elevating grader does not open cuts as they are opened by wheelers or fresnoes. It opens the whole cut at one time, moving back and forth over such a distance as the general contour of the cut indicates to be desirable but always as long as possible up to the full length of the cut. This generates an extension of the haul distance, and the actual haul becomes the distance from the center of mass of the cut as a whole to the center of mass of the fills to

THE generally accepted idea of the usefulness of design is that of an instrument for letting contracts and a guide to be followed in construction. That with a given limiting gradient and rate of curvature, and without change in the quantity of earth moved, the cost of grading with an elevating grader outfit may be varied by as much as 20 per cent, and yet the road produced in each case will be practically the same is a fact quite generally lost sight of.

Again, variation in design may bankrupt a contractor or give him a profit, although, in each case, his contract unit price may be the same and the completed road equally useful and economical in the service of highway transport.

Fluctuations in the length of haul are primarily responsible for high unit costs on elevating-grader projects. Within the well established limits of wagon haul it matters not so much whether the haul be relatively long or relatively short; but when the contractor is put to the necessity of moving a considerable part of the material a long distance and another considerable part a short distance, unit costs for one part or the other are likely to be excessive. The reason is that the adjustment of the size of wagon train to the output of the grader which is appropriate for the long haul can not be appropriate for the short haul, yet practical considerations make difficult the changes in the size of train, which are economically necessary.

In this article the effect of length of haul is treated in detail and illustrations are presented to demonstrate the important bearing of design, involving hauls of various character, on the cost of elevating grader work.

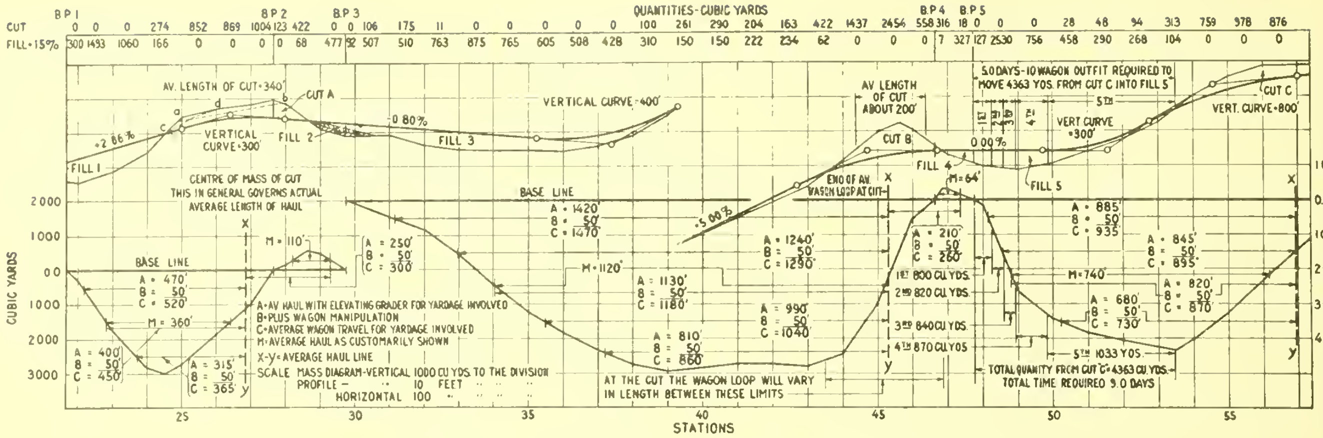
QUANTITIES-CUBIC YARDS
B.P 1
B.P 2
B.P 3
B.P 4
B.P 5
CUT
FILL+15%
AV. LENGTH OF CUT=340'
CUT A
VERTICAL CURVE=300'
FILL 1
FILL 2
FILL 3
+2.86%
-0.80%
VERTICAL CURVE-400'
AV. LENGTH OF CUT ABOUT 200'
CUT B
FILL 4
FILL 5
0.00%
VERT CURVE =300'
CUT C
VERT. CURVE=800'
5.0 DAYS-10 WAGON OUTFIT REQUIRED TO MOVE 4363 YDS. FROM CUT C INTO FILL 5
5TH
CENTRE OF MASS OF CUT THIS IN GENERAL GOVERNS ACTUAL AVERAGE LENGTH OF HAUL
BASE LINE
END OF AV. WAGON LOOP AT CUT
+5.00%
M=110'
M=360'
M=1120'
M=64'
M=740'
A = 470' B = 50' C = 520'
A = 400' B = 50' C = 450'
A = 315' B = 50' C = 365'
A = 250' B = 50' C = 300'
A = 1420' B = 50' C = 1470'
A = 1130' B = 50' C = 1180'
A = 810' B = 50' C = 860'
A = 1240' B = 50' C = 1290'
A = 990' B = 50' C = 1040'
A = 210' B = 50' C = 260'
A = 885' B = 50' C = 935'
A = 845' B = 50' C = 895'
A = 680' B = 50' C = 730'
A = 820' B = 50' C = 870'
1ST 800 CU. YDS.
2ND 820 CU. YDS.
3RD 840 CU. YDS.
4TH 870 CU. YDS.
5TH 1033 YDS.
TOTAL QUANTITY FROM CUT C = 4363 CU. YDS.
TOTAL TIME REQUIRED 5.0 DAYS
A=AV. HAUL WITH ELEVATING GRADER FOR YARDAGE INVOLVED
B=PLUS WAGON MANIPULATION
C=AVERAGE WAGON TRAVEL FOR YARDAGE INVOLVED
M=AVERAGE HAUL AS CUSTOMARILY SHOWN
X-Y=AVERAGE HAUL LINE
SCALE MASS DIAGRAM-VERTICAL 1000 CU. YDS. TO THE DIVISION
PROFILE - 10 FEET
HORIZONTAL 100
AT THE CUT THE WAGON LOOP WILL VARY IN LENGTH BETWEEN THESE LIMITS
CUBIC YARDS
2000
1000
00
1000
2000
3000
4000
0.0
25
30
35
40
45
50
55
STATIONS
x
y

FIG. 2.—Original design of sample grading project, illustrating effect of fluctuating haul distance

which material is delivered. The difference in the two cases is illustrated in the diagram, Figure 1, and in Figure 2 which is the reproduction of the center-line profile of a part of an actual project.

Between balance points 1 and 3 in Figure 2 there is a cut A, which is divided so that part of the material is hauled to each of two fills. The mass diagram which appears below the profile shows that the average haul to fill 1, as commonly calculated, is about 360 feet and to fill 2 about 110 feet.[1] But in taking out material with an elevating grader cuts of this kind will not be divided as they would be for wheeler or fresno work. The grader will work back and forth over the whole length of the cut and the material will be hauled either to fill 1 or to fill 2, with the result that more or less material designated by the engineer as belonging to fill 1 will be placed in fill 2 and vice versa. In a symmetrical cut the result of this practice will be to lengthen the average haul so that it becomes the distance from the center of mass of the cut (line xy as shown on the mass diagram) to the center of mass of the fill. In this way the average haul to fill 1 is raised from 360 to about 400 feet and the average haul to fill 2 from 110 to about 250 feet. A study of the other cuts in Figure 2 shows the same tendency toward an increase in haul distance in all except cut D, all of which is hauled to one fill. Where this occurs no extra haul is generated by the use of the grader.

The result of increasing the haul to the center of mass of the cut as a whole in cut A is more marked in its effect on the haul to fill 2 than to fill 1. This is generally the result where the quantity to be moved in one direction is considerably larger than the quantity to be moved in the other. The haul determined in this way, however, may not be exact. Thus, if cut A had been so nonsymmetrical that the depth of excavation between stations 27 and 29 had considerably exceeded the depth of excavation between stations 25 and 27, the grader might be operated here on a short loop until the material remaining to be taken out was reasonably symmetrical over the whole cut. If this were done the material for fill 2 would be secured with a somewhat shorter haul than the method of calculation described above would indicate.

Similarly, the manner in which even a symmetrical cut is taken out may somewhat affect the length of haul. Thus, if instead of taking a first cut along line ab it is taken along line cd, the haul may be somewhat affected. But in operating elevating graders the general practice is to make the runs as long as possible, and for this reason if the cut is anything like symmetrical the tendency is to make the average haul the distance from the center of mass of the cut to the center of mass of each of the fills or fractional parts of fills into which the cuttings are placed. This should therefore be used as the basis from which to calculate haul. But it should be recognized as an approximate rather than an exact basis, as special conditions of the sort mentioned will often enter to make the actual results somewhat different from the theoretical. Lack of symmetry in the cut is responsible for most of these special considerations. It creates short grader runs, unproductive runs, nonuniform bite, etc., none of which need affect the haul distance as theoretically derived, but all of which do, more or less, because, under field conditions, there generally is less thought given to these matters than is needed to secure accurate results.

EFFECT OF LAYER DUMPING AND WAGON MANIPULATION ON HAUL

Layer dumping also tends to affect haul. If it is required, as it should be, there is a tendency to increase haul at the fill. Thus, in Figure 2, it may be assumed that instead of placing fill 2 as shown on the plans the material secured from cut A is placed in two layers which extend well beyond balance point 3. In such a case the actual average haul instead of being 250 feet might easily reach 300 feet. There is no way of calculating such extensions in advance. They are rather to be considered as an element likely to create excess haul and therefore one which a contractor should watch and avoid. It is not at all necessary that layer dumping cause any considerable amount of extra haul.

Besides these conditions which affect haul there is another arising from the manipulation of the wagons themselves. The general operation of the wagon train, somewhat distorted for the purpose of illustration, is shown in Figure 3. As the elevating grader is moved toward the dump the empty wagon must return to a point some distance back of the loading wagon in order to reach position. Let it be assumed that wagon 10 is loading along ab and wagon 11 is the replacement wagon. Let it also be assumed that the average distance required for picking up a load is 75 feet. The distance that the load as a whole must be hauled will, of course, be calculated from the center of the distance over which the load is picked up. There is then a necessary addition of 37½ feet to the average distance over which the wagon will be moved which addition covers the distance traveled by the

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[1] There are a number of methods of calculating haul. The one here used, which is the simplest and quite accurate enough for the purpose, assumes that the average haul is the distance between the points halfway from the base to the peak of the mass curve on the cut and fill slopes.

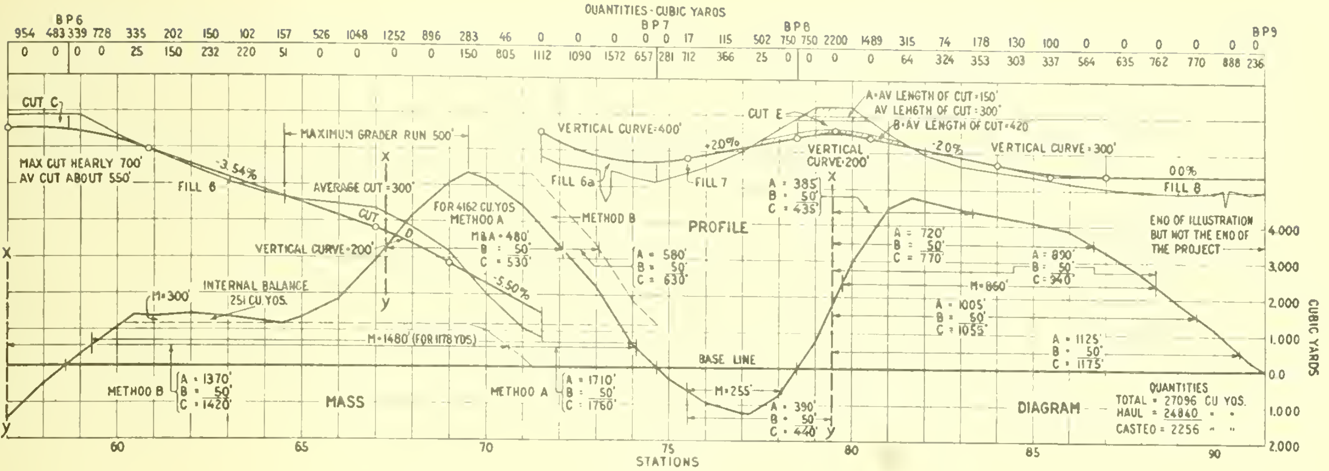

FIG. 2.—(Continued)

wagon before the beginning of the average haul distance for the total load is reached. To this must be added whatever distance was traveled after wagon 11 fell in behind wagon 10 in order to be in a position for prompt replacement Even if the wagon train is in close harmony with the output of the grader, there must still be a short distance allowed for this purpose, say from 10 to 15 feet. If the haul is to fills on both sides of the cut, this extra distance will generally be avoided; but prevailing practice is against taking out cuts in this way, because it doubles the force employed on the dumps. With wagons moving toward the dump, therefore, there is then an excess of wagon travel over the average haul, for the load as a whole, of about 50 feet.

A similar situation prevails when the grader is moving away from the dump. In this case each wagon must move an average of 37½ feet beyond the average haul for the load in order to secure its full load. Moreover, in driving to a clearance for the replacement wagon the loaded wagon must move ahead far enough to permit the replacement wagon to reach the loading position before a turn is made. This will require at least 15 feet and often more. Whether the grader is moving to or from the dump, therefore, the wagons actually travel about 50 feet further than the average haul for the load, the additional travel being necessary in the interest of manipulating the wagons at the grader. Applied to the cut which has been used as an illustration, this results in an average wagon travel of 450 feet to fill 1 and 300 feet to fill 2, with the possibility that when layer dumping is required the haul to fills as short as fill 2 will be somewhat further increased.

WORKING THE WHOLE CUT GENERALLY ADVISABLE DESPITE LONGER HAULS

Ordinarily the contractor will not find it to his interest to avoid the extra wagon travel that arises from the failure to follow the engineer's balance points. With normal efficiency in the operation of the grader and an adequate wagon supply, a cut such as A, which averages about 340 feet in length, permits of working the grader about 50 per cent of the time, but if such a cut were divided into two parts and taken out in accordance with the engineer's balance points the shortened run would permit the grader to work only about 34 per cent of the time in the shorter cut. The resulting loss in output would generally outweigh any advantage that might be obtained from the shortened haul, since it is probable that the wagon time saved by reducing the haul would be wasted in waiting for loads, for it must be remembered that the wagon train generally consists of a fixed number of wagons; therefore, unless the grader output is correspondingly increased, shortening the haul merely tends to cause a piling of the wagons. This is invariably the result when the haul is within the capacity of the wagon train. If the haul is beyond the capacity of the train, the reduction in haul by splitting the cut may overbalance the reduction in grader output, but cases of this sort are comparatively rare, and to take advantage of them the contractor must have at his disposal the accurate data supplied by the mass diagram.

AVERAGE OUTPUT 1,000 CUBIC YARDS A DAY

An analysis of the average wagon travel when combined with an analysis of the average length of each cut, by which the possible output of the grader is indicated, will serve to avoid many of the speculative risks contractors encounter in elevating grader work. To be really effective, however, the analysis should be extended to include a study of the haul cut by cut, and the results of this analysis developed on the basis of some standard unit of production. For this purpose, in the description of the method which follows, an output of 1,000 cubic yards per day has been used.

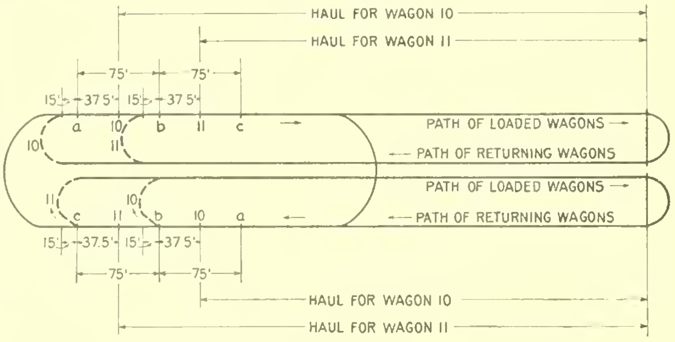

FIG. 3.—Operation of wagon train on each side of grader loop

The bureau's studies of elevating grader projects indicate that the average length of cut is about 450 feet. The studies also indicate that, when the wagon supply is adequate, ordinarily efficient operation results in an exchange of wagons in about 11 seconds. The time required to load a wagon averages about 23 seconds. With due allowance for breakdowns, clean outs, rests, etc., these conditions permit of securing

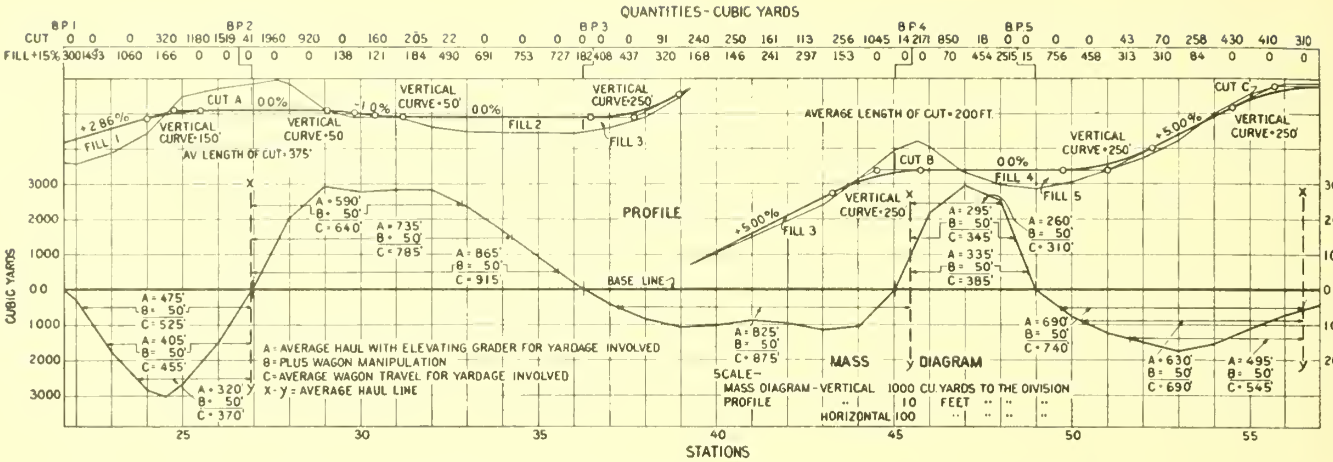

FIG. 4. First re-design of sample grading project, illustrating the possibility of equalizing haul distances. In this design it is assumed that fills will be built entirely from the cuts. This assumption limits the reduction of haul to about half the distance center to center of cuts

an output somewhat over 100 cubic yards an hour with only average efficiency in operation. Allowing, then, for the losses in working time which, on practically all jobs, result in reducing the working day to something under 10 hours, it is still possible for the average contractor, with no change in managerial policies and no improvement in general efficiency, to secure an output of 1,000 yards a day if his wagon supply is adequate and his cuts are of standard length. This output has therefore been used as a basis from which to work in showing how the length of cut and the length of haul are likely to affect output and why contractors often lose money on work of this character through a failure properly to gauge the effect of those elements.

CORRECT BIDDING IMPOSSIBLE WITHOUT STUDY OF HAUL

It will be seen, in cuts A and B, Figure 2, that a knowledge of the average wagon travel does not cover the situation. Thus cut A can be taken out with an average wagon travel of 450 feet for the material moved to fill 1. The mass diagram shows that there are approximately 3,000 cubic yards of material to be moved into this fill. But of this amount 1,000 cubic yards calls for an average wagon haul of about 520 feet, 1,000 cubic yards an average haul of about 450 feet, and 1,000 cubic yards an average haul of 365 feet. In comparison with this the mass diagram for cut B shows about 5,000 cubic yards of material to be placed in fill 3. Of this material 1,000 cubic yards must be moved 1,470 feet, the second 1,000 cubic yards 1,290 feet, the third 1,000 cubic yards 1,180 feet, the fourth 1,040 feet, and the fifth 1,000 cubic yards 860 feet. What is perhaps the most serious problem the contractor has to meet in this sort of work is brought out clearly by the study of these two cuts. An output of 1,000 cubic yards has been set up as the standard day's work, assuming normal efficiency. But if the eight standard days' work in these two cuts are tabulated it will be apparent that the work can not be performed in eight days with any ordinarily constituted outfit, because the range in haul distance is too great. Such a tabulation, in which the effect of length of cut has been purposely omitted, is shown in Table 1.

Table 1 illustrates clearly the dependence of production on the proper adjustment of the wagon train to the length of haul and the danger of depending on the average wagon travel in analyzing elevating grader projects.

TABLE 1.—*Effect of length of wagon haul on elevating grader production and time required for grading*

(Based on cuts A and B, Figure 2)

Material moved to fill	Amount of material	Wagon haul	Number of wagons required for full output of grader	Using 8-wagon outfit		Probable daily output	Time required to move each 1,000 cubic yards	Equivalent yardage [1]
				Team time wasted	Grader time wasted			
	Cubic yards	*Feet*		*Per cent*	*Per cent*	*Cubic yards*	*Days*	*Cubic yards*
1	1,000	520	8	0.0	0.0	1,000	1.0	1,000
1	1,000	450	7	12.5	0.0	1,000	1.0	1,000
1	1,000	365	6	25.0	0.0	1,000	1.0	1,000
3	1,000	1,470	16	....	50.0	500	2.0	2,000
3	1,000	1,290	15	....	47.0	530	1.9	1,887
3	1,000	1,180	13	....	38.0	620	1.6	1,613
3	1,000	1,040	12	....	33.0	670	1.5	1,500
3	1,000	860	10	....	20.0	800	1.2	1,250

[1] Based on standard daily production of 1,000 cubic yards.

Time required for movement of 8,000 cubic yards, 11.2 days; probable average output per day with 8-wagon outfit, 720 cubic yards.

Cuts A and B are on one project and only a short distance apart. The average wagon travel from cut A to fill 1 is 450 feet, and from cut B to fill 3 it is 1,180 feet. These averages would indicate that trains of 7 and 13 wagons, respectively, would be required for 100 per cent production at the grader as against the range of from 6 to 16 wagons shown by the detailed analysis actually to be required if full production is to be secured. If an 8-wagon outfit were used, 100 per cent grader production could be obtained in cut A, but the longer hauls of cut B would reduce the average daily output for both cuts by 28 per cent. Even if the long wagon train indicated by the average travel to fill 3 were provided, full production at the grader still could not be maintained at all times. The point to be stressed here is that haul varies over such a wide range that no contractor is in a position to make other than a random guess as to what the work on any project will cost without a careful analysis of this element.

THE MASS DIAGRAM AS A MEANS OF CONTROLLING FIELD OPERATIONS

The analysis of elevating grader work through study of the mass diagram can be made an effective means of controlling field operations. Thus, cut E is a symmetrical cut from which approximately 6,000 cubic yards of material are secured. Wagon haul per 1,000

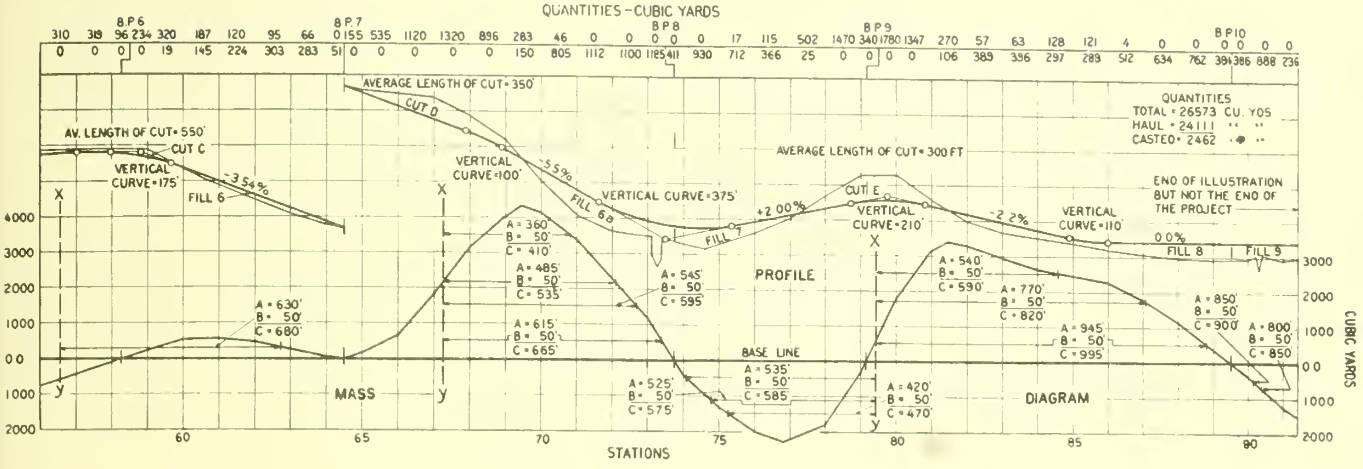

FIG. 4.—(Continued)

cubic-yard unit to fill 8 varies from 435 to 1,175 feet. But this cut offers a grader loop varying in average length from 150 feet for the top three feet to over 420 feet at the bottom. At the top the grader loop will be short and the production low. If the top of this cut is placed at the far end of the fill, the low output at the grader and the long haul tend to offset each other, with the result that only 12 wagons will be required in order to maintain an adequate supply at the grader. If this is done, the bottom of the cut will fall at the near end of the fill, and for placing it here 7 wagons will be required. If this process is reversed, the top of the cut being placed at the near end of the fill, only 6 wagons can be used, while in placing the material from the bottom of the cut at the far end of the fill 16 wagons could be utilized. If the wagon train consists of, say, 10 wagons, 30 per cent of the wagon time would be lost in placing the close-in material, and the grader would be idle about 17 per cent of the time in placing material at the far end of the fill under the first scheme of operation. If the second scheme of operation were adopted, 40 per cent of the wagons would be idle while the material close in was being placed, and the grader would be idle about 37 per cent of the time while material was being placed at the far end of the fill. The advantage of the first arrangement is clear. There is no way of determining points of this kind by rule of thumb. Therefore the field superintendent often overlooks them. They are clear enough, however, when the mass diagram and the profile are thoughtfully scrutinized.

This is only one of the many illustrations which could be given to show that the mass diagram on work of this kind offers a logical means of controlling field operations, just as it offers the proper basis on which to study a project prior to selecting the outfit best suited to economical work on it. Another illustration of the usefulness of the mass diagram in controlling elevating grader work is found in fills 6 and 6a (Fig. 2), which are built from cuts C and D. In this case the mass diagram indicates the movement of approximately 5,300 cubic yards of material between balance points 6 and 7. There is a considerable excess of material—about 1,200 cubic yards—originating in cut C which must be hauled to some point beyond cut D. There are approximately 250 cubic yards in a small fill between cuts C and D, which for the purpose in hand is of no consequence, and some 4,100 cubic yards originating in cut D which must be placed in fill 6a. The mass diagram indicates an average haul of 480 feet for this latter material. If to this is added 50 feet for wagon manipulation, the average wagon travel becomes 530 feet. As against this the average wagon travel for the 1,178 cubic yards originating in cut C is 1,760 feet, if this material is hauled to the far end of fill 6a. If, however, this material is placed at the near end of fill 6a the wagon haul is reduced to 1,420 feet, and by so doing the average wagon travel on the material taken from cut D is extended to 630 feet. This illustration shows that contractors can often improve output by studies of the details of the work. At current bid prices on this sort of work (about 25 cents per cubic yard) the gain in daily production with a 10-wagon outfit would be worth nearly $16, as shown by Table 2, which is a saving well worth while.

TABLE 2.—*Comparison of production under two methods of working cuts C and D (Fig. 2) with a 10-wagon outfit (average management)*

Material from cut	Quantity	Scheme A			Days work required	Scheme B			Days work required
		Haul	Wagons required for 100 per cent production	Production with 10 wagons		Haul	Wagons required for 100 per cent production	Production with 10 wagons	
	Cubic yards	*Feet*	*Number*	*Per cent*	*Number*	*Feet*	*Number*	*Per cent*	*Number*
C	1,178	1,760	23	44	2.7	1,410	19	53	2.2
D	822	660	9	100	0.8	755	10	100	0.8
D	1,000	600	9	100	1.0	685	9	100	1.0
D	1,000	500	8	100	1.0	620	9	100	1.0
D	1,000	415	7	100	1.0	525	8	100	1.0
D	350	325	6	100	0.3	470	7	100	0.3
Total	5,350				6.8				6.3

Average daily production:	Cubic yards
Scheme B	850
Scheme A	787
Gain	63

At 25 cents per cubic yard, value of increased daily output—63×.25=$15.75.

A wide-awake superintendent can sometimes save team time by using a double dump. This is of advantage only when a fill contains yardage that is within the haul limit of his outfit and other yardage that is outside of this limit. A 10-team outfit, with average management, can keep the grader busy up to a wagon-travel distance varying from 600 to 850 feet, depending on the length of cut from which the loads are being secured. As an illustration, suppose the mass diagram

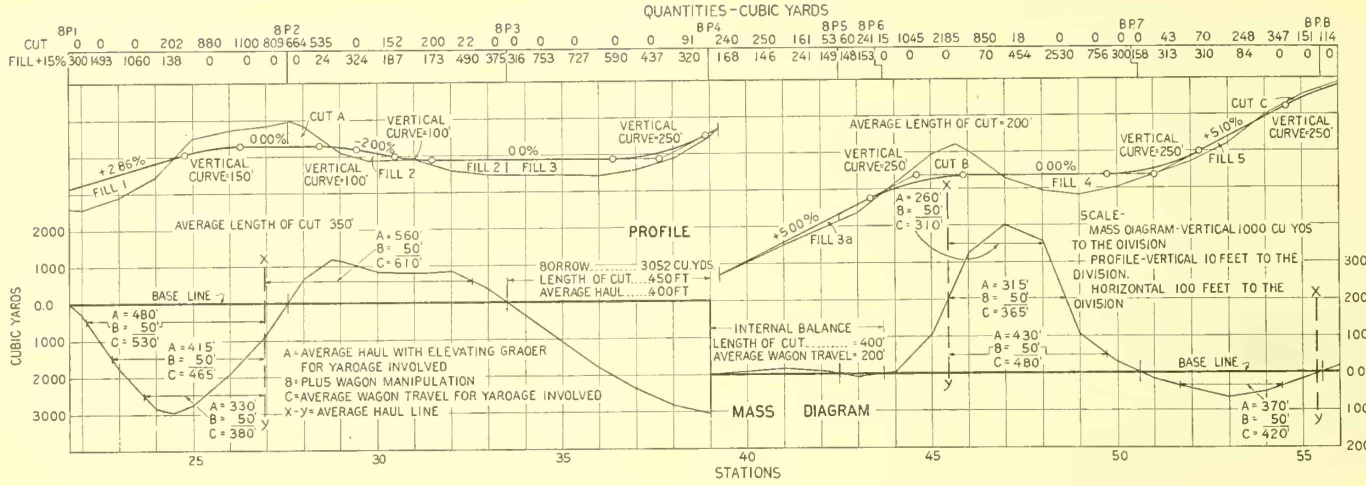

FIG. 5.—Second re-design of sample grading project, illustrating how hauls may be further reduced by use of borrow pits

indicates that the current day's work involves an average wagon travel of 350 feet. Six wagons are required to keep the grader busy, and if eight are sent out the time of two will be wasted. If under such conditions enough wagons are diverted to a longer haul, so that the proper rate of wagon supply is maintained, the time of the two wagons can be saved. A study of the time-distance graph for wagons (the time-distance graph will be discussed in the third part of this article) will show how many wagons must be diverted if this method of operation is to be successful. When this is done, however, the superintendent must use considerable care to insure that the long-haul wagons do not become bunched, as bunching will largely obliterate the benefit to be derived. The dumping at the long-haul dump need not be given much supervision as it may be so handled that subsequent operations will cover work of this kind and correct any careless placement of material.

The mass diagram may also be made to give a picture of each day's work and so to assist the contractor by showing how much material he has placed. How this can be done is shown in the mass diagram for fill 5 (Fig. 2). Once the mass diagram is in hand all that the contractor needs to know is the distance completed in order to determine the yardage he has placed with considerable accuracy. This is a valuable control and one that should not be overlooked.

POOR DESIGN REDUCES ELEVATING GRADER OUTPUT

The foregoing illustrations indicate some of the problems a contractor faces in his effort to operate an elevating grader outfit with maximum efficiency. They show that the mass diagram is a valuable guide to proper bidding and also in the conduct of the work. But they also show clearly that the highest degree of economy may be impossible unless the road is designed with that end in view. The profile shown in Figure 2 represents a decidedly poor design for elevating grader construction. The essence of economical design for this type of equipment is that the cuts and fills shall be so related in yardage and in the distance between them that a reasonable wagon train can keep the elevating grader operating at or near full capacity all the time. This means that variations in haul should be kept within a reasonable range. A design which produces a succession of long and short hauls, such as are found in fills 1, 2, 3, and 4 of Figure 2, violates the first principle of economical elevating grader operation: First, because the short cuts reduce the rate of output of the grader; second, because some of the hauls are so long that no reasonable wagon train can handle the full output of the grader; and third, because other hauls are so short that the wagon train can work but a fraction of the time with the grader working at full speed. More nearly correct treatments of this section of highway are illustrated in the redesigns shown in Figures 4 and 5.

In preparing these modified designs it has been assumed that local conditions govern the height of the fills. These have, therefore, been left approximately at the same height as in the original design, and the modification in the grade line has been limited to a readjustment of the cuts in order to secure more nearly uniform hauls. Cut depths have been freely altered and vertical curves have been standardized by the adoption of a uniform radius of curvature of approximately 5,000 feet. No particular effort has been made to keep the yardage the same as in the original design, although it is approximately the same. The purpose is to illustrate certain principles of design, the employment of which is not in any way dependent upon the yardage.

The object in presenting the redesign shown in Figure 4 is to lay emphasis on the manner in which by careful design the haul on elevating grader projects can be so adjusted as to improve output and reduce production cost. In this first redesign it has been assumed that the fills will be built entirely from the cuts. Obviously this assumption limits the reduction of the maximum haul to about half the distance from center to center of cuts, yet even with this limitation it is possible materially to reduce the wagon travel. Under the original design only 42 per cent of the material could be moved with a wagon travel of less than 600 feet, and 35 per cent involved a travel of over 900 feet. Under the first redesign (Fig. 4) 59 per cent of the material can be handled with a wagon travel of less than 600 feet and only 8 per cent requires a travel of over 900 feet. The haul of this latter material is imposed by the distance between cuts, and as already stated it can not be reduced under the system of design in use.

PRODUCTION COST REDUCED 11 PER CENT BY FIRST REDESIGN

That the first redesign is an improvement from the standpoint of the contractor will be apparent without much explanation. Under the redesign economical operation requires nearly as many wagons as under the

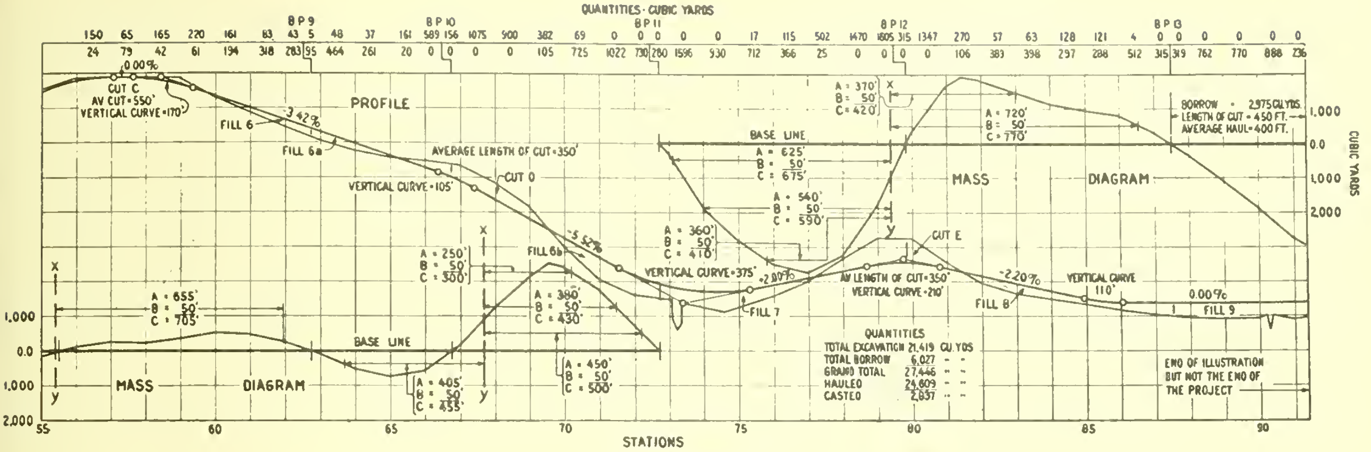

FIG. 5.—(Continued)

original design, but because much less work has to be done over the long wagon travel distances the cost of production is reduced about 11 per cent. At the prevailing bid price for work of this kind (25 cents per cubic yard) this difference, if obtainable on projects generally, should in the long run mean a reduction of from 2 to 2½ cents per cubic yard in the price paid by the State for elevating grader work. Whether on any large percentage of projects a similar saving could be made by redesign has not been fully investigated. This is, however, of less importance to the designer than the fact that it can be made on some projects.

To the contractor such a saving is of vital concern for his money is at stake. To him the original design and the redesign present a concrete example of how design often affects his profit by introducing a wide range of wagon travel; and the comparison of the two designs should explain why two jobs in which topographic conditions are similar may return different percentages of profit or loss. The writer has more than once been told by distracted contractors that it has been impossible to obtain proper production on the job in hand though jobs in similar territory had on other occasions yielded a fair profit; but never has there appeared to be any appreciation of the fact that the fault might lie in the design rather than in the way in which the job had been handled or that the size of the outfit might have affected the cost of handling the work.

BORROW PITS REDUCE COST BY ELIMINATING LONGER HAULS

Figure 5 shows a second redesign. Under this design long haul has been further reduced by the use of borrow pits. In the Mississippi Valley it is often possible to obtain stripping rights, i. e., the right to take material to a certain depth, generally a foot or two, from abutting property. The advantage of this practice lies in the fact that the grader run can readily be made such as to yield 100 per cent production at the grader, while at the same time the haul can be kept within the capacity of a normal wagon train.

Whether to use the method or not, from the standpoint of the State, depends on the saving likely to result and the cost of the rights. From the standpoint of the contractor its desirability is apparent as its use reduces the longer hauls imposed in the first redesign by the distance between cuts. Under this design only seven wagons would be required for the most economical prosecution of the work, 87 per cent of the material would be hauled less than 600 feet, no material would require wagon travel in excess of 900 feet, and the production cost would be 20 per cent less than under the original design. A condensed comparison of the three designs is given in Table 3; the methods used in deriving the results shown will be explained in Part 3 of this series of articles.

TABLE 3.—*Comparison of cost of three designs using various wagon trains*

Design	Figure No.	Total actual yardage	6-wagon outfit		7-wagon outfit		8-wagon outfit	
			Equivalent yardage [1]	Production cost	Equivalent yardage [1]	Production cost	Equivalent yardage [1]	Production cost
		Cubic yards	*Cubic yards*	*Cents*	*Cubic yards*	*Cents*	*Cubic yards*	*Cents*
Original	2	27, 096	--------	--------	--------	--------	--------	--------
First redesign	4	26, 573	--------	--------	--------	--------	31, 480	14. 3
Second redesign	5	27, 446	33, 663	13. 5	30, 445	12. 7	29, 419	12. 9

Design	9-wagon outfit		10-wagon outfit		11-wagon outfit		12-wagon outfit	
	Equivalent yardage [1]	Production cost	Equivalent yardage [1]	Production cost	Equivalent yardage [1]	Production cost	Equivalent yardage [1]	Production cost
	Cubic yards	*Cents*	*Cubic yards*	*Cents*	*Cubic yards*	*Cents*	*Cubic yards*	*Cents*
Original	--------	--------	33, 177	15. 9	31, 612	15. 8	30, 644	15. 9
First redesign	29, 887	14. 1	28, 947	14. 2	--------	--------	--------	--------

[1] Based on standard daily production of 1,000 cubic yards.

Minimum production cost:
Original design, 15.8 cents with 11 wagons.
First redesign, 14.1 cents with 9 wagons.
Second redesign, 12.7 cents with 7 wagons.
Difference in production cost=3.1 cents, or about 20 per cent.

PRODUCTION COST IN RELATION TO HAUL

The relation of haul to cost may be further illustrated by a very general production cost statement. Under the wage scales prevailing in the Mississippi Valley and contiguous territory the fixed costs of operating an elevating grader are not far from $80 a day. The cost of hauling is about $5 per team per day. These costs cover field pay roll, feed for the teams, cookhouse losses, and minor repairs only and, for the purposes in hand, will be referred to as the production cost. The contractor has many other costs to meet, such as the costs of office overhead, bond, financing, getting onto the job, depreciation, etc., all of which are presumably included in the bid price, but these are largely independent of production cost and should not be included, since it is only the factors affecting production cost that are under consideration.

Accepting the above production costs as a basis of comparison and assuming the condition of an elevating grader working in a 450-foot cut and the wagon supply necessary for standard production of 1,000 cubic yards a day at the various lengths of wagon travel, the costs of production per cubic yard will be as shown in Table 4.

Table 4.—*Unit costs of production for standard production from a 450-foot cut with various hauls*

Average wagon travel to fill	Wagons required	Daily production cost per outfit	Production cost per cubic yard
Feet	*Number*	*Dollars*	*Cents*
325	6	110	11
500	8	120	12
675	10	130	13
850	12	140	14
1,025	14	150	15

Average rate of increase, 0.6 cent per station.

The table presents, of course, a highly generalized statement of the costs, but it will serve to show, for example, that if the wagon travel can be kept down to a point where an 8-wagon outfit can be appropriately used, production cost will be about a cent a yard less (bid prices should average about 2 cents less) than where the wagon travel is such that a 10-wagon outfit is required. If, with any given wagon supply, the length of cut is shortened the effect will be to lower production so that the saving resulting from a shortening of the haul may in this way be offset by the reduced output obtainable.

If, on the other hand, the wagon supply is not adequate, the cost will mount rapidly. Thus, if an 8-wagon outfit (basic cost of production 12 cents) is working where the wagon travel is 1,025 feet, it will supply only eight-fourteenths of the necessary number of wagons; the production will fall, therefore, to eight-fourteenths of normal, and the cost of production will rise to 21 cents.

Two important points should be somewhat clarified by this analysis. The first of these is that the contractor who undertakes a project with less than the proper wagon supply will generally find that his production cost is higher than the haul distances prevailing would normally generate; and the second, that the injection of occasional long hauls has relatively a more important effect on production cost than a general increase in the haul for which provision can be made in the selecting of the outfit.

To the contractor this latter point is of special importance in that it shows that bids on overhaul must be carefully scrutinized. It is customary to take overhaul on work of this sort at about 2 cents per station yard. If the station yardage of overhaul is large, the contractor will be justified in providing a wagon supply sufficient to care for it properly, and the price of 2 cents may be sufficient or even excessive, but more often the overhaul is generated by a few cuts from which long hauls are required. In such cases it is not practicable to increase the wagon supply and the output may be so reduced, when operating on this long-haul work, that the production cost alone may about equal the price received. In such cases the contractor loses on his overhaul.

SUPPLEMENTARY USE OF FRESNO MAY SAVE MONEY

In the foregoing discussion it has been assumed that contractors use an outfit of fixed size. This, in fact, is the all-but-universal practice. Under this system team time is lost in considerable amounts whenever the wagon travel is short, and grader time is lost whenever a long travel distance is encountered. There is, however, another method of handling such work that deserves more consideration than it is now receiving.

It is well known that the fresno can move dirt on short-haul jobs about as cheaply as it can be moved by an elevating grader outfit. This being the case, there seems to be no good reason why the fresno and the elevating grader can not be combined with profit when the conditions warrant. There are a good many angles to this question, and it is quite impossible to treat all of them in the short space here available. Briefly, however, the situation is substantially this: The elevating grader is a wonderfully effective loading mechanism; the wagon is perhaps the most efficient hauling mechanism available for work within its proper field. As long as these can be kept in balance and efficiently operated, the cost of producing yardage in place is probably as low as it can be made with any type of earth-moving equipment, particularly if the haul is of any considerable length. However, as the haul approaches zero there is a short distance, probably not exceeding 200 feet, in which the fresno can produce yardage in place about as cheaply as the elevating grader. For such short hauls the wagon supply of the elevating grader must be excessive, since it is properly designed for the longer hauls. On the average project there is always a certain amount of work to do which is well within the field of the fresno, and there are hauls which, while clearly within the field of the elevating grader, are shorter than the average for the project and are capable of handling with less than the full outfit of teams. If, whenever this condition obtains, the teams not needed by the grader are shifted to fresnoes, whatever yardage is moved by the latter will represent a clear gain in production.

The field of competition again turns against the fresno when the distance becomes so short that the elevating grader can cast the material into place, assuming, of course, that no wagons are kept idle during the casting operation and that the ditches are not so deep that the output of the grader will be sharply reduced by the tilting of the machine. But the fact that is apt to be overlooked in considering the relative cost of short haul and casting is that so long as the contractor must maintain his teams (including drivers) he can gain nothing by casting unless he thereby increases his output per hour. The possibility of increasing output by casting lies in the fact that the time lost in wagon exchange is saved, but this saving is not always a net gain, because a certain amount of time must always be lost in resting the grader stock. As a matter of fact, when the grader is drawn by a tractor or by 20 horses the output generally is somewhat increased when casting, but if only 16 horses are used the aggregate of the rest periods is generally about as great as the time lost in wagon exchange, and no particular advantage accrues.

Under these general conditions if a contractor, desiring a high average rate of output at the lowest possible cost, instead of selecting a wagon train on the basis of the average haul would select it on the basis of the longer hauls and then plan to send out with the wagons each day only those teams which can be worked to capacity, using the balance in taking out ditches and short-haul cuts with fresnoes, he would find that under practically all circumstances his grader could be worked to capacity, and his extra teams, instead of spending much of the time waiting to be loaded, would be producing yardage at a profitable margin. In discussing this scheme of operation with contractors two objections to it have been raised: First, that teamsters operating wagons do not like to transfer to fresnoes and, second, that the standard 1½-yard wagon requires only two horses while the 4-foot fresno requires three.

There is a valid answer to both objections. To the first there is the answer that employers generally find no difficulty in enforcing conditions which are clearly set forth when men are employed. The other may be met with the blunt statement that the 1½-yard wagon has no place on elevating grader work and should be replaced by the more efficient 2-yard wagon. The 2-yard wagon requires three horses, but day in and day out it will haul 50 per cent more than the 1½-yard wagon. Where it is used there will be no difficulty in shifting the 3-horse teams to 4-foot fresnoes.

TWO-YARD VERSUS ONE AND ONE-HALF YARD WAGONS

The desirability of using 2-yard instead of 1½-yard wagons needs no very extended defense. In the first place, as noted above, 50 per cent more yardage is secured per load at the expense of only one extra horse. At the present time it costs about 70 cents each per day to maintain stock. With drivers at $3.50 the cost to a contractor of maintaining a 2-horse team on the job is in the neighborhood of $4.90 a day. As compared with this, the cost of maintaining a 3-horse team is about $5.60. Thus, with a 14 per cent increase in cost a 50 per cent larger load can be moved, and this is an advantage that no contractor can afford to overlook. Moreover, the actual load per horse is slightly less when three horses are used on a 2-yard wagon than when two horses are used on a 1½-yard wagon, because while the pay load per horse is about the same, there is no important difference in the internal frictional resistance of the two sizes of wagons.

In addition to this saving in the hauling cost, which is a relatively large one, the larger normal grader output must also be considered. This larger output is due to the smaller number of wagon-exchange periods for a given yardage. As by its use only about two-thirds as many waits between loads are required, the 2-yard wagon, used with normal efficiency should produce at least 10 per cent more output than the 1½-yard wagon.

The one valid objection which has been raised to the use of the larger wagon is that in soft ground it tends to mire down a little more than the 1½-yard wagon. This can be avoided by supplying the larger wagon with a tire of proper width.

It is impossible to recommend the use of wagons larger than the 2-yard size, because when more than three horses are used there is difficulty in getting under the belt quickly. There is also some difficulty in maneuvering at the dump. These problems are not, however, of any consequence where three horses are used, and the evident success of those outfits now using 2-yard wagons offers concrete evidence that the advantages here noted are being secured by at least a few progressive contractors.

The natural deduction from these facts is that if the contractor desires to operate as profitably as possible, and at the same time to reduce the element of risk in his elevating grader work to a minimum, he will study the various elements of his job with care and will provide enough wagons that he can keep his grader working at capacity. There are industries which to-day make all of their profit out of the use of materials formerly wasted. The situation of the elevating grader contractor is somewhat analagous in that as his work is now conducted, team time is wasted in large amounts. One way of utilizing waste team time has been mentioned. Others could, no doubt, be suggested. The point is that every time a team stands idle when it could be made to produce something value is lost. Salvaging this value will prove profitable to any contractor who will undertake it seriously and methodically. If, to his efforts to salvage lost time he will add a serious study of those elements in this sort of work which are responsible for its present speculative aspects, particularly the element of haul, he should have little trouble in avoiding the financial troubles so often encountered by those operating in this field.

(Continued from p. 58)

that maintenance costs on macadam roads are approximately one-sixth as much per mile per vehicle as those on gravel roads, and in individual cases much less than this amount.

Even estimating 1,000 vehicles per day instead of 500 as the capacity of a gravel road, reference to the 1930 forecast map (Fig. 10) will indicate that a considerable mileage of gravel roads on the Maine primary system should be reconstructed with more durable surfaces within the next six years.

On the basis of a maximum capacity of 1,000 vehicles per day for gravel-surfaced highways, the following program for the improvement of Maine highways is suggested for the period 1925 to 1930:

1. Construction of high-type pavements on the heavy-traffic routes.

2. The heavy-traffic routes included in the suggested improvement program, listed below, are divided into three groups based on density of traffic, the type of traffic on each highway, and the urgency of the need for immediate improvement.

3. It is suggested that the routes in Group I be improved first, those in Group II second, and those in Group III last.

4. Because of the greater total traffic as well as the larger number of motor trucks per day on the highways in Group I, high-type pavements are suggested for this group.

5. On the basis of Maine construction and maintenance experience it is believed that bituminous macadam, of the type now being constructed in the State will adequately serve the present and future traffic on

the highways included in Groups II and III for the expected life of the bituminous-macadam type of construction.

6. In the selection of the surface type for highways in Groups II and III consideration should be given to the present type of surface on sections of the highways included in these groups.

7. Following is a description of the highways included in each group:

Primary system

Group I:
- Kittery—Portland—Brunswick.
- Portland—Auburn—Augusta.
- Augusta—Gardiner.
- Waterville—Fairfield.
- Bangor—Oldtown.

Group II:
- Brunswick—Gardiner.
- Brunswick—Belfast.
- Brunswick—Auburn.
- Augusta—Waterville.
- Fairfield—Bangor via Newport.
- Bangor—Ellsworth.
- Waterville—Oakland.

Group III:
- Belfast—Bangor.
- Bangor—Ellsworth via Orland.
- Ellsworth—Bar Harbor.
- Fairfield—Skowhegan.
- Wells—Berwick.
- Portland—Bridgton—State line.
- Gray—Norway.
- Auburn—Farmington—Strong.
- Newport—Dover.
- Perry—Calais.
- Houlton—Presque Isle—Van Buren.

Secondary system

Group I:
- Portland—Westbrook.

Group II:
- Auburn—Mechanic Falls.
- Wells—Sanford.

Budget requirements for this period can be established by computing the mileage of each type to be constructed (total mileage of designated routes less improvements already made) and estimating the costs of such construction. If estimated available revenues over the period are not sufficient to meet such expenditures, the possibility of a bond issue to be retired from funds derived from an increased gasoline tax is suggested.

It is estimated that a gasoline tax of 1 cent per gallon will yield approximately $4,500,000 during the six-year period 1925 to 1930, inclusive. An increase of this tax from 1 to 3 cents per gallon would provide approximately $9,000,000 in additional revenue.

During this same six-year period license fees, assuming no change in fees, may be expected to yield approximately $17,000,000, making a total of $21,500,000 from license fees and a gasoline tax of 1 cent per gallon, or a total of approximately $30,000,000 from license fees and a gasoline tax of 3 cents per gallon.

NEW TESTING DEVICES DEVELOPED

Three new testing devices for the use of the highway engineer have been developed by the Division of Tests of the Bureau of Public Roads.

A test for the consistency of concrete in the field has been devised as a substitute for the slump test, which is not particularly reliable under all conditions. The method is particularly adapted for concrete paving and other work in which a relatively dry consistency can be employed. It is based upon the principle that, within working limits, the consistency of freshly mixed concrete is proportional to the weight which will be retained upon a plate of given diameter when the concrete is deposited on it in a standard manner. The device consists of a truncated cone large enough to hold about 75 pounds of wet concrete, supported by an angle-iron frame above a circular plate 15 inches in diameter, which in turn rests upon a spring balance.

The method of testing is as follows: The apparatus is placed upon the subgrade and the cone is filled with concrete immediately after the batch has been deposited by the bucket. Immediately after filling the cone a removable slide at the bottom is withdrawn and the concrete flows out upon the plate. If the mix is either very dry or very wet, a larger quantity will roll or flow over the edges of the plate than if it is moderately dry. The plate is supported above the spring balance by two cams which take the weight of the concrete off the spring until it has all been deposited. By turning a handle which revolves the supporting cams the concrete upon the plate can then be weighed. It has been found that for a 15-inch plate the usual variations in the amount of water in a 1 : 2 : 3 paving mix will cause a difference in weight of from 20 to 50 pounds. Experiments have indicated that for machine-finished work the proper consistency to use is one which will give the greatest weight of concrete retained upon the plate. For hand-finished work a mix slightly wetter than this would probably have to be used. This device has been tried on actual construction and it appears to be of practical value. A more complete description of the process, together with test data illustrating the use of the apparatus, will appear in an early issue of PUBLIC ROADS.

Work has also been carried forward on a device designed to register the intensity of pressure used during the molding of Portland cement mortar briquettes. Considerable latitude has always been allowed operators in regard to this detail of cement testing, resulting in quite appreciable variations in manipulation, which undoubtedly affect test results. The device consists of a small weighing platform approximately 15 inches long and 4 inches wide, large enough to hold a glass plate and a three-gang briquette mold. The device is so arranged that an electric contact is made when a certain pressure is exerted on the briquette by the operator, lighting a white light in the front of the apparatus. Another contact, which when made shows a red light, may be set for a pressure beyond which the operator should not go. In determining whether an operator is using the proper pressure it is only necessary for him to mold a set of briquettes on the weighing platform and to note whether the pressure he exerts is sufficient to light the white light but not the red light. Both of the contact points are adjustable so that any pressure within working limits may be recorded.

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ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.

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1279. Rural Highway Mileage, Incomes and Expenditures, 1921 and 1922.

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Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

APRIL 30, 1925

GOVERNMENT PRINTING OFFICE

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1925									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1924			PROJECTS COMPLETED SINCE JUNE 30, 1924			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 4,599,721.63	$ 2,186.247.54	464.1	$ 1.211.826.12	$ 597,175.34	132.2	$ 15.535.183.95	$ 7.536.992.14	919.8	$ 83,897.90	$ 41,949.95	0.5	$ 2,432,469.03	Alabama
Arizona	8.338,365.41	4,287.683.88	527.8	954.635.99	553.824.61	77.5	2.003,975.73	1,214,965.20	141.4	751,503.67	478.400.86	55.6	2,016.997.45	Arizona
Arkansas	11.094,751.31	4,424.345.63	944.4	2.027.459.11	861.846.28	89.2	6,696,593.84	2.982.172.11	362.3	582.115.38	247,945.55	33.4	1.910,254.43	Arkansas
California	12,999.075.03	5,647.148.17	533.7	8,111,073.28	4.377,480.41	320.1	9,468.989.20	4.491,514.94	231.9	235.597.25	137.985.74	11.1	4.911.912.74	California
Colorado	8,108,070.31	4.029.998.97	502.6	2,485,167.72	1.327.686.59	108.3	4.681.519.97	2.484,092.77	193.1	434,324.88	156.517.34	9.9	2.934.922.33	Colorado
Connecticut	3,062,872.02	1,269,658.60	73.6	344,021.87	163,383.00	8.2	3,144,249.93	995,572.55	50.8				1,427,481.85	Connecticut
Delaware	3,056,832.22	1,007.714.83	72.5	1.089.734.63	431,625.92	30.8	638,530.64	270,431.10	16.1	30.647.50	15.323.75		380,059.50	Delaware
Florida	961,134.07	461,470.92	48.8	1,998,139.65	944.017.05	47.5	8.633.082.35	4.202,091.68	236.9	116.547.09	91,547.09	0.1	1.480,638.26	Florida
Georgia	17,167,373.32	7,955,806.20	1,214.2	2,826,450.71	1,370,881.74	260.9	10,768,861.15	5,310,312.75	720.1	649,058.00	287,629.36	25.1	1.508.356.95	Georgia
Idaho	8,181.697.92	4.092.395.52	506.8	1.048,443.42	597.683.81	70.2	1.982.767.55	1.238,144.41	136.9	647.468.63	430,955.79	53.4	1.255,458.47	Idaho
Illinois	26,964.706.06	12.279,546.33	804.7	12.087,130.27	5.902.126.18	397.1	5.987.993.79	2.873,742.27	202.4	2,519.871.45	1.255.092.37	90.0	4.317.463.85	Illinois
Indiana	7,577,444.16	3,655,540.97	225.7	4,844,133.70	2,334,801.41	159.2	12,317,347.93	6,010,171.02	399.9	594,281.83	297,140.96	20.3	3.953,430.64	Indiana
Iowa	23,195,778.19	9,237.031.86	1,682.9	4.020.893.35	1,842,654.30	290.3	6.030.879.18	2.307,655.17	376.8	2.920.333.58	1.231.126.05	99.9	2,788.065.62	Iowa
Kansas	17.084,136.48	6,043,176.80	502.7	8.526.612.68	3.347.336.40	294.2	12.963.047.64	5.585,975.49	604.9	1.220.170.43	353,734.05	42.8	2.043,426.26	Kansas
Kentucky	10,822,980.31	4,613,947.28	429.4	3,068,981.16	1,238,955.61	119.5	7,974,588.37	3,625.924.83	298.2	448.259.93	161,246.58	4.7	2.143,271.70	Kentucky
Louisiana	8.488.463.18	3,636,143.36	661.2	2.506.645.04	1.216,178.85	218.8	4.141,634.04	2.023,616.02	199.9	120.174.25	60,087.12	10.0	1.326,678.65	Louisiana
Maine	6.911.058.78	3,299,935.38	230.7	1.263,222.53	507.934.95	50.7	749,878.17	350.363.39	24.2				1.516,978.28	Maine
Maryland	8,760,044.42	3,213,321.78	243.2	1,267,148.78	590,561.24	48.1	1,784,838.61	768,190.52	53.7	919,150.96	357,131.89	28.2	355.527.57	Maryland
Massachusetts	10,191,202.02	4.105.727.22	232.8	3.138,337.19	1,132.910.86	56.6	4.195,527.34	1.232.855.80	67.6	945,862.58	244,699.88	13.9	2,293,704.24	Massachusetts
Michigan	13.434,135.07	6,060,612.23	494.5	2.165.559.55	994.055.92	101.6	14,965,650.26	7.044.686.58	479.5	163.170.00	81,585.00	5.4	3,924,069.27	Michigan
Minnesota	24,037,561.24	9,885,843.07	2,292.0	6,378,124.65	2,852,798.97	429.2	6,325,341.64	2,928,600.00	667.0	647,775.08	41,300.00	63.3	1,734,027.96	Minnesota
Mississippi	7.988.193.89	3.828.941.39	655.0	2.122.301.02	1,021.625.74	148.2	8.350,578.28	4,167,448.04	482.6	14.353.65	7,176.82		1,798,041.01	Mississippi
Missouri	11.352,027.70	5.245.899.18	803.5	5.266.914.59	2,601.319.53	264.6	24,189,720.35	9.774,930.05	807.7	3.663.762.32	1,204.095.22	119.3	1,531,671.02	Missouri
Montana	8,867,279.16	4,384,335.12	791.4	1,289,321.25	933,188.03	130.2	1,861,844.92	1,367,818.50	158.6	1,062,089.80	603,378.69	119.9	5,226,169.66	Montana
Nebraska	7.876,337.16	3,714,691.59	1,440.4	1.138.077.45	534.606.07	66.0	7,933,588.33	3.929,316.09	887.4	491.629.53	245,725.94	40.7	4,608,575.31	Nebraska
Nevada	3,460.245.52	1,853.624.98	225.6	1,446,557.71	1,228,225.07	131.4	4.431,686.34	3,731,975.54	366.6	245.675.33	209,952.34	33.8	814.619.07	Nevada
New Hampshire	3,076,750.19	1,487,867.58	171.3	1,088,937.67	498,359.29	36.8	760,383.53	366,326.09	25.0	63,256.58	30.502.65	1.7	417.533.39	New Hampshire
New Jersey	7,623,795.12	2.661,531.49	148.7	3.925,528.07	1.049.793.50	63.0	7,922,198.08	2.406.081.64	49.1	136.173.65	44,160.00	2.9	1.363.762.37	New Jersey
New Mexico	5,306,286.45	2.758.849.68	714.3	2.682,242.02	1.712.033.39	300.2	5.125.836.15	3,350.646.06	461.4	250,993.39	154.354.77	16.7	1.798.614.10	New Mexico
New York	18,862,742.49	8,257,844.44	572.7	6,200,578.11	2,506,693.04	165.3	27,131,862.84	10,002,819.38	593.0	7,289,400.00	2,000,967.50	131.8	7,596,919.64	New York
North Carolina	12,567.732.97	5.676,757.66	884.7	6.093.681.22	2.189,158.07	169.8	8,154.431.12	3.350.994.58	219.3	2.354.066.81	1.052.808.73	56.2	1.723.709.96	North Carolina
North Dakota	9,088,973.11	4,418.505.42	1,587.9	1.707,265.90	833.931.18	325.6	2.628.753.74	1.269.985.04	355.3	706.957.94	339.035.59	79.9	2.622.896.77	North Dakota
Ohio	33,122,751.43	11,879,917.99	962.5	7,996,678.33	3,212,875.94	216.0	8,617,837.99	3,021,034.95	258.3	2,649,183.76	1,132,091.70	75.7	3,683,831.42	Ohio
Oklahoma	12.986.865.26	5,888.852.03	497.3	6,891.839.01	3.351.322.64	303.1	6.647.061.76	2.606.096.78	244.7	2.013.951.44	883.639.88	95.0	1.561,896.67	Oklahoma
Oregon	12.082.873.17	5.819.093.79	655.6	2,162,545.40	1.274.243.08	140.2	2,464.973.77	1,419.566.88	105.7	196.470.25	119.432.94	15.6	1,053,490.31	Oregon
Pennsylvania	36,825,248.98	14,114,694.79	729.7	4,110,864.64	1,505,005.07	84.5	20,659,418.64	5,410,193.75	336.0	17,649,575.35	4,978,197.18	347.3	1,953,648.21	Pennsylvania
Rhode Island	1,774.397.25	779.227.96	46.0	854,098.95	340,460.13	18.8	1,411.044.89	406,599.59	18.9	475.198.35	132,090.00	8.8	640.288.32	Rhode Island
South Carolina	9,016.476.73	4.124.045.22	924.4	1.822,999.07	846.189.44	280.6	6.299.660.52	2,372.182.32	396.6	725,276.32	282,968.56	14.8	1,114.709.46	South Carolina
South Dakota	8.674.597.86	4.244.636.27	989.8	3.367,308.41	1.720.478.53	447.8	5,733.499.27	2.808.687.34	791.9	979.062.92	458,497.57	135.8	701.400.29	South Dakota
Tennessee	6.805.683.35	3,313.936.07	259.6	5.808,932.39	2.830.881.08	198.0	12,803.570.05	5,794,277.31	453.1	208.868.55	104.434.27	9.5	1,604,093.27	Tennessee
Texas	42.341.999.56	16,190.624.91	3,122.8	10,675,000.47	4,381.946.83	683.4	23.868,278.36	10.004.692.83	1,470.3	3.631.658.94	1.430,267.76	205.8	4.132,495.67	Texas
Utah	3.304.423.75	1.895.805.92	219.0	1.787.404.63	1.156,002.09	158.6	3.901.513.02	2,572.253.79	230.4	474.143.39	336,183.74	50.7	1,002,694.46	Utah
Vermont	1,922.114.16	942,769.12	74.4	809,908.56	384,869.27	23.8	1,461.197.20	698.908.78	35.4	10.699.53	5,349.76		867.707.07	Vermont
Virginia	10.035.301.48	4.801.782.43	562.5	2.609.879.62	1.253,191.20	99.7	9.678.136.61	4.392,276.66	336.9	1.633,065.17	735.085.81	53.4	860.329.90	Virginia
Washington	11,384,615.67	5,290,895.45	457.0	1,759,239.46	739,794.59	66.2	2,855,248.30	1,371,400.00	120.6	296,296.71	134,000.00	9.0	1,469.574.96	Washington
West Virginia	5.489.747.96	2.365.041.53	255.6	1,853.452.91	865.251.80	71.1	4,166,291.65	1,791.537.78	127.8	2,458,182.33	900.745.56	63.9	628.850.33	West Virginia
Wisconsin	18.753,903.15	7.441.033.57	1.325.3	3,001.359.02	1,452.667.68	121.0	2,317,433.76	1,096,311.36	109.7	878,417.26	462,708.00	40.7	5,099.038.39	Wisconsin
Wyoming	6,127,625.61	3,078,098.70	687.6	2,629,027.49	1,626,626.97	290.5	2,736,020.87	1,702,079.25	122.2	310,162.08	199,661.00	14.0	1.015.840.08	Wyoming
Hawaii										342,377.22	97.440.00	6.5	633.810.00	Hawaii
TOTALS	549,655,391.27	237,852,399.82	32,452.9	162,465,684.77	75,335,658.59	8,314.6	353,002,451.62	156,564,394.12	15,847.9	65,240,138.95	24,316,350.31	2,317.0	104,181,197.16	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 91,290,453.71 Federal aid $ 40,220,630.47 Miles 3,616.7

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A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 6, NO. 4 ▽ JUNE, 1925

Sampling the Subgrade of the Ohio Post Road

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A JOURNAL OF HIGHWAY RESEARCH

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THE PUBLICATION OF RESEARCH[1]

By E. W. ALLEN, Chief, Office of Experiment Stations, United States Department of Agriculture

THE ultimate aim of research is publication. It may be deferred, but it is due eventually if the research has been successful. To some this final task, like the end of a poor cigar, is often very bitter. But in research the end is even more important than the beginning, and quite as deserving of being well done as any other part of the work.

The idea that in writing a paper "anything will do," and that it is a weakness and a waste of time to be fussy about it, will rarely bring a creditable printed report or stamp the author as a painstaking investigator. Writing is, to some extent, a special aptitude or acquirement, and those with whom it is not a native ability can do much to improve it by practice and by taking pains. It is especially important for the scientist, because it is his means of communication and, to a large extent, of his influence. Hence his attitude toward the task should be right.

Publication is a serious matter because of the permanence of the record. A printed paper can not be recalled or retracted as the spoken work can; it stands for all time. Of course, a statement may be explained or modified after it has been published, but it is difficult to reach all readers of the original, and the correction may be overlooked in future citation.

For us in the Department of Agriculture, which stands so close to the public on the one hand, and to various branches of science on the other, the obligation to publish the results of investigation in suitable form is no less heavy than that of making the work itself exact in method and deduction. The manner in which this is done will reflect not only upon the individual worker but upon his organization and the department as a whole.

Unfortunately, much scientific writing of the present time is loose and indefinite in its expression, verbose often to the point of being tedious, and out of harmony with the ordered, exact, and logical nature of science itself. It is the subject of much unfavorable comment not only by the press but by other classes of writers. Sir James Barrie recently remarked that "The man of science appears to be the only man who does not know how to say it." Apropos of certain "diffuse" and "overloaded" papers presented at the British Association a few years ago, the London Times pointedly remarked that "Science should not disdain the art of presentation." The publications of the department and the experiment stations have long been popular subjects for newspaper gibes, sometimes partially merited and frequently a reflection on the reporter for his lack of understanding or for being so far behind the times.

While we can never hope wholly to escape such comment, we can take away such ground for it as lies in faulty writing or failure to adapt the text to the audience addressed. For it is all too true that scientific men are often unsuccessful writers, not only for popular reading but for their fellow specialists. This is partly because their training has not been in that line and their minds have been schooled to analytical habits, and partly because they are not willing to take sufficient pains or exercise critical judgment in selecting and presenting their material.

The purpose of writing is not only to express ideas, but to communicate them to others. Science is not inherently dull, heavy, and hard to comprehend; it is essentially fascinating, understandable, and full of charm. It is simple, after it has been worked out, and is capable of being stated in concise terms easily understood. But to succeed in conveying ideas correctly and in a readable way requires considerable effort on the part of most of us. It calls for time to do it well. It is just as important as making more experiments, although the worker may not like it as well, and is quite as worthy of his best effort. He should take care not to overestimate his ability to dash off a research paper at odd times.

CLEAR THINKING MAKES CLEAR WRITING

The aim in publishing research, as well as in carrying it on, is to leave the field clearer than you found it. If that can not be done it is doubtful whether a scientific paper is justified. There can not be clear writing without clear thinking, and when one learns to write clearly he will in the process learn to think clearly. Indeed, it may be doubted whether thought and its expression can be separated. Vagueness or turbidity of language usually indicates similar qualities in the thinking. The attempt to express a matter clearly in writing thus helps in the process of clear thinking. Bacon wrote that "reading maketh a full man, conference a ready man, and writing an exact man."

Since the object of writing is to communicate information, the writer may well give special attention to being intelligible. The audience addressed needs to be kept in mind and the language adapted to the reader. Sir Clifford Allbutt lays down the good rule to "take pains with yourself first, then with your reader"; and says further: "A writer who writes to convince and not merely to see his name in print must learn to lay his mind alongside that of his reader."

In other words, it is necessary to understand and keep in mind the point of view of those it is desired to reach, the mental background with which the new facts must be harmonized. The writer must know how to present his facts and arguments so that they will fit into the reader's experience and what he already knows. The reader may know something about the subject, but he doesn't know the point of view from which it was taken up, the purpose, and the reasoning, or how the work further clarifies the subject, unless these things are presented in their proper setting.

Clearness is absolutely essential in technical writing. It is not enough to use language that may be understood—it is necessary to use language that can not be misunderstood. Whether we agree with the author or not, we should never be in doubt as to what he means.

One of the first requisites to clearness of expression is mastery of the language—not in the mere passive sense of avoiding errors, but positively, as a flexible medium for the exact, unmistakable expression of thought The choice of words, the order in which they are arranged, the sequence of clauses composing sentences, and, finally, the arrangement of sentences in a paragraph, are important features in effecting clearness.

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[1] A lecture delivered before the class in "The Nature and Method of Research," of the graduate school, Department of Agriculture.

Words are the vehicle of language. They are to impart ideas; hence care needs to be exercised in their choice. There are fine shades of meaning to be observed. The use of the right word will save considerable explanation and thus assist brevity. "The wrong word derails the thought; the needless word is an obstruction." The cultivation of a fairly broad vocabulary is desirable, but it is preferable to repeat the same word, if necessary to make the meaning clear, instead of adopting one that does not fit, for the sake of variety.

Lavoisier, writing on the expression of ideas in 1789, said:

> Every branch of physical science must consist of three things—the series of facts which are the objects of science, the ideas which represent these facts, and the words by which these ideas are expressed. Like three impressions of the same seal, the word ought to produce the idea, and the idea to be a picture of the fact. * * * As long as precise terms are lacking we can only communicate false or imperfect impressions of these ideas to others.

Having something to say, therefore, say it in your own way, provided you use good diction, the right word, and a simple form of expression. Above all, make your meaning clear. Read over each sentence to see if it expresses what you desire to say. Eliminate each word that is not necessary to the sense or the spirit of the article. Words are only useful for expressing ideas; fine writing and high-sounding phrases have no place in technical articles. Choose your words with care. Make each sentence convey an idea, and don't try to put more than one idea in a sentence. Punctuate so as to bring out your meaning; the punctuation is a part of the writing.

On this subject of lucidity in writing, I should like, even at the risk of some repetition, to quote from Dr. Erwin F. Smith, of the Department of Agriculture, who has given some remarkably good advice on writing.[2] He says:

> Clarity is the soul of truth, and especially in science there should be an idea behind every expression, and this idea should be stated as clearly as language permits. * * * There are various ways of saying things, but only one best way. Nevertheless, to read the contributions of many scientific men one would suppose they must think any method of expression sufficient, even the most clumsy and ambiguous. Yet such is not the case. In spite of this motley array of bad writers, it is best that subject and predicate should agree, * * * and especially that each statement should be susceptible of but one interpretation.
>
> Every paragraph and sentence in your paper should receive careful and repeated consideration, first, as to whether it tells the exact truth; second, as to whether it is absolutely clear, i. e., will convey the same meaning to all as to yourself; third, as to whether it is complete, or requires various additions or qualifications (science is an eternal qualification); fourth, as to whether the sentences in it are entirely logical and move convincingly toward your final conclusions. These things can be determined only by repeated readings and much pondering. * * * Occasionally there is a person who can write a thing as it should be the first time trying, but I have known only one or two such persons. Generally, easy writing is hard reading. Darwin sometimes recast his paragraphs a dozen times, and most of us may expect to reach a good style, if at all, only by dint of much labor and repeated rewriting. Yet who can doubt that it is an end worth all it may cost?

USE THE SIMPLEST WORDS THE SUBJECT WILL BEAR AND NOT TOO MANY OF THEM

So remember the reader. Be sympathetic toward him. He must make some effort, but he is not bound to follow you through. The writer has not the same hold on his audience that the speaker has. You may have to see it through if you get into a dull or unprofitable lecture, but the reader does not have to stay by an article that is hard to follow, badly put together, or tedious in its extravagant length. The author must interest and hold the reader if he expects to accomplish his purpose in publication. If a paper is direct and understandable, and has something worth while to say, it will be readable and interesting—often entertaining. If it is not interesting it will not be read, or only skimmed as a matter of duty. It is not always the fault of the public or of brother scientists if they are not familiar with your published work; a part of the burden rests on you.

Study to communicate the results of research in a way that will involve the least effort on the part of the reader to take them in. Spencer said that "Those are the most effective modes of expression which absorb the smallest amount of the recipient's attention in interpreting the symbols of thought, leaving the greatest amount for the thought itself." Technical articles naturally require rather close attention in reading, but it is poor writing when a sentence or a passage must be reread two or three times to get at its meaning. Writing that leaves the reader's mind in such a condition that it can uninterruptedy follow the meaning of the paper without being conscious of the words has been described as good style. Huxley's idea of style was "to say that which has to be said in such language that you can stand cross-examination on each word."

Brevity is another important quality of a technical paper. This does not mean that the presentation should not be adequate to a clear understanding of what is reported and ability of the reader to judge the merits of the contribution; but the length should be proportionate to the actual contribution. Nowhere are more skill and judgment required.

A publication is not judged by its length but by the message it carries. The amount of well-directed and well-digested work it represents, the discrimination shown in distinguishing between the important and the relatively unimportant, and conciseness in presenting the essential features are its claim to recognition.

Verbosity and diffuseness suggest a possible lack of really important matter to fill out the space, or at least that the writer has not digested what he has to say. As one writer puts it: "Verbosity is a sign either of carelessness or of a lack of time to take care."

The question of what to include will be determined to some extent by the character of the publication; but whether it is a technical or a semipopular one, the question of what to leave out will be one for very careful consideration, which frequently can not be settled at the first writing. On review it may be found that considerable may be left out without sacrificing anything really essential. Descriptions and statements of facts gain force by brevity and by sticking quite closely to the real kernel of the subject.

As a rule, the more definitely a fact has been established by an investigation, the more directly and simply it can be presented. It is the doubtful ones that have to be hedged about with explanations, qualifications, and cautions.

On this subject of brevity Doctor Smith says:

> A good rule is never to use two pages for a subject that can be compressed by a little thinking into one. The generality of men use more words to express an idea than are actually necessary, if the best words had been chosen. Study the meaning of words, their shades of meaning, and rewrite a subject twenty times, if necessary, to state it cogently and with brevity. Remember nearly everybody will read a brief statement on an interesting subject, while only the most phlegmatic

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[2] Bacterial Diseases of Plants, by E. F. Smith, pp. 643–647.

and determined will hold themselves to a long-winded one. You will more than treble the number of your readers by halving your paper.

Morever, for the necessity of those who can't spend even the minimum of time necessary to read a short paper, and for the convenience of everybody, especially of the foreigner, it is your solemn duty to sum up the substance of your contribution in a series of brief conclusions which everyone will read, and which, if well put, may induce many to turn back and read your whole paper.

The style of the technical paper should be simple, straightforward, and dignified. It should suggest neither a fairy tale, a sensational newspaper story, nor a sermon, but rather a simple, unaffected, and uncolored account of work done and its application. Accuracy and clearness ought never to be sacrificed to a supposedly more popular style. The presentation should be such as to win the reader's confidence in the thoroughness and reliability of the work reported.

Accuracy of fact and statement is, of course, a primary requirement in technical writing. Carelessness is intolerable, a reflection on the author which can not be shifted to the stenographer or the printer, even though these may be found at fault. The investigator who is careless in reporting his work lays himself open to the suspicion that he may be careless in his experiments, in taking data, and even in thinking about them.

PLAN BEFORE YOU WRITE

With these things in mind, what should be the method of preparing for publication? I have no new receipt, no suggestion that has not been made by others, but with this acknowledgment I may attempt to put into form some of the essentials we have been discussing.

In presenting a scientific paper or a bulletin the author says, by inference at least: "Here is the product of my labors upon this subject, taking advantage of what others have done and building upon that foundation. I present it because the work has reached a stage which marks an advancement worth recording. I give it to you in a simple and straightforward form which will enable you to understand the status of the subject, my contribution to it, and its bearing or application. It is the product of my investigation and my mature judgment. I am ready to stand by my work and to be judged by this account of it."

Schopenhauer divided writers into three groups. The first and largest group wrote without thinking; the second thought and wrote at the same time; and the third group, a very small one, thought before they even began to write.

Outline or plan.—It is a good idea, therefore, to begin with the making of an outline or plan to be followed in constructing the article. Order is the first law of nature; hence an account of research dealing with a study of nature ought to reflect this quality in its arrangement. One of the first requirements is that the paper should be systematic, orderly, and logical in its method of presentation, progressive in its effect, so that the course can be followed and interest will cumulate to the very end.

The title.—First in order will come the title. This can not always best be determined at the beginning, but it has to be provided for, and so may be considered here. It is a matter of much more importance that it is sometimes thought to be. Of course there is a limit to what can be put into a title, but it should be specific and descriptive as far as it goes. Bibliographers complain bitterly of misleading or incomplete titles, and as bibliographies, abstracts, and indexes must be depended on for finding a paper after it is a few years old, an inadequate title is one means of burying a piece of research.

There have been several articles in Science recently on the proper wording of titles of scientific papers, especially from the standpoint of the librarian and bibliographer. In one of these an earnest plea was made for such clear and definite titles as will enable the accurate cataloguing of articles, so that the investigator must find them in his search for the literature.

The introduction.—Next will come the introduction, which, in an account of research, would set forth the problem and give a brief review of the recent literature bearing on the subject. Such a review should rarely constitute a monograph of all that can be found, as is sometimes the case, but be sufficient to show the status when the writer entered the field and the place where his work began. This will naturally make clear what the specific object of the research was—whether the adding of a new fact or idea, or a further interpretation.

This introductory matter will usually constitute a relatively small part of the article and be confined to what is strictly pertinent to the subject under discussion. It should not suggest what the old lady referred to as a *preramble.* Dr. E. E. Slosson, of Science Service, gives the following advice in this connection: "Don't back up too far to get a running start. Remember the man who wanted to jump over a hill. He ran a mile to work up momentum and was so tired when he got to the bottom of the hill that he had to sit down and rest. So will your readers be. Ninety per cent of the manuscript that I have handled in 20 years as an editor would have been improved by cutting off the first page or paragraph. Yet authors, like hens, kick on decapitation."

Body of the article.—We are now ready for the account of the investigation itself. Begin at the beginning, and let the account unfold itself in a systematic, logical way, as a verbal account of some experience would, but with the possibility of doing even better. When we are talking we often revise a somewhat confused statement by "that is to say" or "in other words," which indicates dissatisfaction with the way the matter has been put, and that in translating his thoughts into words the speaker has clarified his own thinking. This correction will usually be taken care of in the revision.

The reader will naturally be interested in a brief statement of the plan of procedure, the method employed, an indication of the scope of the study, and conditions under which it was done. The account will aim to show that the writer had a clear purpose in starting the work, realized what he was going after in his investigations, has the facts arranged in his own mind, and has studied them so that he can impart them to others and draw warranted deductions and conclusions.

REMEMBER THE READER

To quote Doctor Slosson again:

> Don't overestimate the reader's knowledge, and don't underestimate the reader's intelligence He may not know as much as you do about this particular thing—let's hope not, anyway—but otherwise he may be as bright as you are—let's hope so, anyway.
>
> Don't forget that your reader is interrupting you every 10 lines to ask "Why?" "What for?" or "Well, what of it?" and if you don't answer his tacit questions he will soon stop reading.
>
> Don't shoot in the air. Aim at somebody. You may miss him, but you are more likely to hit somebody else than if you aim at nobody. Look out of your window and note the first person coming along the street. Imagine yourself stopping the man or woman on the sidewalk and, like the ancient mariner, holding his or her attention till you have told your tale to the end.

Analyze the subject carefully and break it up into parts. Use center heads and side heads in the text to divide it and express the relation of parts. This helps in the progressive arrangement and assists the reader to follow. Paragraph freely, separating different points in this way and bringing together things which belong together. This makes the page look less solid and makes reading easier.

As far as possible finish each topic or phase of the subject as you go along. Do not revert to it further on in the text to add some new point. This makes confusion. When comparisons are made between different parts of the paper, refresh the reader's mind sufficiently so that he will not necessarily have to turn back and reread in order to understand the new point.

Data and tabular matter.—A preliminary step toward writing is the preparation of the data. It is often possible to work up the data as the investigation progresses from one stage to another. This is helpful in connection with deciding on the next step. Before writing, the results will need to be critically reviewed to determine what they show and how they can best be arranged. The arrangement of data is an important part of the task of presenting the matter to someone else.

This occasion will be one for sifting, condensing, and summarizing. It is a selective process. I know there are those who contend for the publication of all the important data, so that readers may not only follow the author's reasoning but be able to check up critically on his deductions. Some writers insist on a practical reproduction of their notebooks, but this seems rarely warranted or necessary. Liberality should be exercised, of course, but the permissible liberality is determined in no small measure by the character of the article. The main object is to record matters which have permanent value, confessedly a matter of good judgment, in which there is perhaps as much danger of overestimating the importance of details as of leaving out too much.

The object of a table is to present a picture of the data, as complete in itself as possible. The purpose of tabulated data is not alone to record it, but to clarify the subject—to present the matter more clearly and concisely than it could be presented in the text. A table is a difficult thing for many people to make and for others to understand, particularly if it is unduly complicated or improperly constructed. The reader should be prepared for it by a text which leads up to it with a simple and direct explanation and a suitable title which tells what it is about. There is often room for improvement in such headings. Sometimes tables are designated merely by number. Do not cover too many different points in a single table. This makes the table complicated and leads to confusion, and it often results in including things which must be referred back to in the discussion several pages beyond. Long tables are apt to be confusing.

Try to avoid putting different units in the same column, as pounds, tons, dollars, etc. Sometimes this is necessary, but usually it can be avoided by turning the table about. As far as possible, tables ought to be self-explanatory, but sometimes the different treatments or variables are indicated by letters or numerals requiring reference to the text to supply the information, which usually is difficult. The table ought to clearly indicate the essential variables.

Don't forget that the reader will expect the author to make some explanation of what he considers a table shows, or how it prepares for what is to follow. It was not unusual in the past to "let the results speak for themselves," presenting them with no attempt at comments or comparison. Frequently this reflected a difficulty of interpretation, and so raised the question as to why such data should be inserted if the author himself was not able to make anything out of them.

THE SCYLLA OF OVERSTATEMENT AND THE CHARYBDIS OF UNNECESSARY QUALIFICATION

Avoid qualifying unnecessarily or too extensively. The aim should be to make fairly definite statements regarding the teachings of the investigation. These may be limited in their scope, but they ought to be direct, so that they can be understood. This is true whether they are conclusions, generalizations, or tentative suggestions. It is important to make the meaning clear, to be cautious and reserved, keeping well within the facts; but there is such a thing as being too cautious and too reserved, as if the author were not sure what he believed. It is a good rule to say what you have to say so people will give you credit for your suggestion or deduction; if not, it will be difficult to establish claim to originality later when some one else makes a positive announcement.

Illustrations.—A good illustration is often the quickest way to convey an idea. Frequently it will do the work of a whole page of description. The same is true of the presentation of results in graphic form, provided such graphs are not so complicated as to require an undue amount of study. The object of illustrations is to illuminate the text but not to embellish it—to make it more intelligible or to give a more definite impression. There is sometimes a tendency to overillustrate, which is a decided disadvantage in a scientific article. Aside from the expense this entails, it is confusing to the reader to be constantly running upon illustrations which are naturally supposed to have some bearing on the text, but after some waste of time are found not to have. Where possible there should be a reference to each figure or illustration in the text, and each figure should have a legend or title, telling what it illustrates and explaining it where necessary.

Conclusions.—The reader will expect some conclusions. He may be suspicious if there are too many or if they are too far-reaching. The author here has the opportunity of crystallizing the results and showing how they clarify the subject. The important points developed in the investigation should be brought together, with such deductions, suggestions, or generalizations as seem warranted. The accurate drawing and stating of these conclusions is one of the most delicate steps in preparing a scientific paper, requiring not only caution and discrimination but unusual care in wording to avoid possible misunderstanding.

After the paper has been entirely written, revise it clearly for the plan and method of presentation, and for the form of statement. This will frequently result in shortening the paper and making it more direct in its treatment, and it will give opportunity for the weighing of each statement for accuracy and clearness. If possible have some one else read it who has technical knowledge of the subject. Prefer criticism before rather than after publication. It is usually more agreeable.

Most institutions or organizations now have provision for some editorial review of the manuscript before it is sent to the printer. The editor's task is a delicate one. It calls for tact, sympathy, and patience, but he is in a position to render a real service to writers of technical papers, and his efforts should be welcomed and given respectful consideration.

THE WAGON AND THE ELEVATING GRADER

AN ECONOMIC STUDY OF THE WAGON-ELEVATING GRADER COMBINATION, IN THREE PARTS

BY THE DIVISION OF CONTROL, U. S. BUREAU OF PUBLIC ROADS

Reported by J. L. HARRISON, Highway Engineer

PART III.—ESTIMATING THE COST OF ELEVATING-GRADER WORK

THE output of the elevating grader as used in highway construction and the importance of the attendant haul have been discussed in previous articles of this series. This article deals with the wagon supply and shows the application of the data secured to a few of the practical problems of the engineer and the contractor, particularly those encountered in preparing bids and controlling going projects.

As in the study of the other forms of earth-moving equipment, the determination of the number of wagons required is made on the basis of a time-distance graph, the pitch of which shows the rate at which the wagons are moved. In its general characteristics the typical time-distance graph for standard 1½ cubic-yard wagons is similar to those for the wheeler and fresno, but its pitch is higher than that of the fresno graph and not so high as that of the wheeler graph. Dynamometer studies are being conducted by the Bureau of Public Roads for the purpose, among others, of analyzing the reasons for this difference. While these studies have not yet advanced far enough to yield conclusive data, they appear to show that the power required to haul loaded wagons is less than that required to haul loaded fresnoes and greater than that required to haul loaded wheel scrapers. This presumption seems to be confirmed by the greater apparent variation in the rate at which wagons operate than in the rate at which either wheelers or fresnoes operate. In one or two instances, jobs have yielded a wagon time-distance graph indicating a rate of travel nearly as high as that commonly maintained by wheel scrapers. On the other hand, wagon jobs not infrequently show a rate of travel somewhat under that here presented as standard. This suggests that if the soil and gradient are favorable, the resulting light draft tends to bring the wagon speed up to the wheeler speed, while unfavorable soil conditions or adverse gradients increase the power required of the teams with a corresponding tendency to decrease the rate of travel.

Low rates of wagon travel appear also to be associated with a low rate of movement by the elevating grader. If the elevating grader is operated at less than its proper speed (3¾ feet per second) it is necessary to rein in the wagon teams while the load is being put on and they are apt to hold to the slower pace when moving with the load. The remedy is in efficient superintendence.

The pitch of the standard wagon graph, Figure 1, indicates a normal travel speed of approximately 245 feet a minute, or about 4 feet a second. The rate tends to be a little higher on the return empty than on the loaded haul. In estimating elevating grader work, however, the normal rate may be used unless the physical conditions prevailing on the job are such as to indicate that the hauling will be unusually difficult.

METHODS OF DETERMINING WAGON SUPPLY REQUIRED

For use in estimating the number of wagons required Tables 1 and 2 have been prepared. Table 1 shows the number of loads per hour which each wagon should move at the above normal speed for various lengths of haul. Table 2, among other things, shows the output of the grader, with average management for various lengths of cut. With these data, the number of wagons that should be supplied for any given length of wagon travel can be readily determined. For example, assuming normal operation in a 600-foot cut with an average haul distance of 700 feet, we find from Table 2 that the possible grader output in a 600-foot cut is 86 loads per hour and from Table 1 that a wagon normally operates at the rate of 8 trips per hour on a 700-foot haul. It follows, therefore, that eleven 1½ cubic-yard wagons will be required to supply the grader.

The same determination can be made graphically by use of the graph presented in Figure 2. Following the vertical line representing the 600-foot cut until it intersects the horizontal line for the 700-foot length of haul, we find that the intersection falls between the curves for 10 and 11 wagons, respectively. Whenever the determination yields a fraction of a wagon greater than one-half an additional wagon should be supplied, except that if the number of wagons indicated is five or less, an additional wagon should be supplied if the fraction is larger than a quarter. This is desirable because too much trotting of the

ON ELEVATING-GRADER jobs variation in the length of cut and the wagon haul from the cut to the fill introduces an element of risk which is not present in grading jobs on which fresnoes or wheelers are to be used.

The fact that the earth is excavated by one unit and hauled by another makes the economical planning and performance of the work a somewhat complicated and difficult matter. Unless there is proper adjustment of the two units one to the other, so that the grader will excavate enough material to keep the wagon train busy, and, *per contra*, that a wagon will be in position for a load whenever the grader is ready to load it, there is a loss in production which, if frequently repeated, may consume the contractor's profit.

The quantity of earth the grader will excavate in a day depends upon a number of conditions which have been discussed in detail in a previous article; but one of the principal governing factors is the length of cut, since the time consumed in turning the grader is necessarily unproductive. The number of wagons required to keep the grader busy depends upon the length of haul.

On all jobs there is some variation in the lengths of cuts and hauls, and it is not practicable, therefore, to plan the work so as to obtain maximum efficiency of the wagons and grader at all times. But for any job there is some best size of wagon train which will reduce the cost of the work to a practicable minimum.

This article describes a method by which, for any particular job, the economic size of wagon train can be determined; and the probable cost can be estimated with a minimum of risk.

TABLE 1.—*Number of loads per wagon per hour for various haul distances*

Haul distance	Time per round trip	Number of trips per hour
Feet	*Minutes*	
100	2.57	23.3
200	3.39	17.7
300	4.21	14.2
400	5.03	11.9
500	5.85	10.2
600	6.66	9.0
700	7.48	8.0
800	8.30	7.2
900	9.12	6.6
1,000	9.94	6.0
1,200	11.58	5.2
1,400	13.22	4.5
1,600	14.85	4.0
1,800	16.49	3.6
2,000	18.13	3.3

TABLE 2.—*The effect of length of cut on percentage of time grader is nominally at work. Average operation with adequate supply of 1½ cubic-yard wagons*

[Loading distance, 75 feet; loading time, 23 seconds; exchange time, 11 seconds; grader turning time, 45 seconds; grader delays, 10 per cent]

Number of loads	Length of cut	Wagon-loading time	Average wagon exchange	Grader-turning time	Time losses due to breakdowns, rests, etc. (10 per cent)	Total time	Loads per hour	Percentage of time grader is at work	Percentage of production on basis of 450-foot cut and adequate wagon supply
	Feet	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Number*	*Per ct.*	*Per ct.*
1	75	23	--------	45	7	75	48	30.6	58.0
2	150	46	11	45	10	112	64	41.1	78.0
3	225	69	22	45	14	150	72	46.0	87.2
4	300	92	33	45	17	187	77	49.2	93.4
5	375	115	44	45	20	224	80	51.4	97.4
6	450	138	55	45	24	262	82	52.7	100.0
7	525	161	66	45	27	299	84	53.8	102.0
8	600	184	77	45	31	337	86	54.6	103.6
9	675	207	88	45	34	374	87	55.3	105.0
10	750	230	99	45	37	411	88	56.0	106.2
15	1,125	345	154	45	54	598	90	57.7	109.3
20	1,500	460	209	45	71	785	92	58.6	111.1

N. B.—A loading loop consists of a run down the cut, a turn, a run up the cut and a second turn. In developing this table a half loop—one run and one turn—has been used. By this device the "length of cut" may be compared directly with plans for construction work. Output, working time, etc., are the same as they would be if a whole loop had been used.

stock would be necessary if only five wagons are required to make up an average deficiency as great as a full half wagon, which is 10 per cent of the total wagon supply. While a well-managed wagon string can safely carry a 10 per cent overload for a considerable period, it is easier to operate smoothly on a 10 per cent underload, and unless the superintendence is of a high order the loss in output which is likely to result from the attempt to carry an overload will more than counterbalance the saving in cost of team time.

Figure 3 shows the wagon supply required under average management if 2 cubic-yard wagons are used instead of the 1½ cubic-yard wagons assumed in Figure 2. These graphs are based on the rate of wagon travel shown in Figure 1 and on the conditions noted on the graphs.

Often an outfit will be operated somewhat above average efficiency in one or more of the details most important in governing production. For such outfits the wagon supply should be somewhat larger than is indicated in Figures 2 and 3. To cover such cases the graphs shown in Figures 4 and 5 have been prepared, and from them may be secured the information for use in estimating the wagon supply required when increased efficiency is secured from whatever cause. Line A gives data for average management. Line X gives data for a 5-second saving in total time per load; lines Y and Z, for savings of 10 seconds and 15 seconds, respectively, while line B is for extra-good management, where about 16 seconds are saved. Figure 4 is for use with the graphs in Figure 2 when wagons of 1½ cubic yards capacity are used, and Figure 5 with Figure 3 when wagons of 2 cubic yards capacity are used. The conditions under average management as contrasted with extra-good management, shown as line B in Figures 4 and 5, are given in the following table:

Item	Unit	1½-cubic-yard wagons		2-cubic-yard wagons	
		Average management	Extra good management	Average management	Extra good management
Loading distance	Feet	75	60	107	90
Loading time	Seconds	23	16	35	24
Exchange time	do	11	5	11	5
Grader turning time	do	45	45	45	45
Grader delays	Per cent	10	5	10	5
Grader speed	Feet per second	3.26	3.75	3.26	3.75
Wagon speed	do	4.08	4.08	4.08	4.08

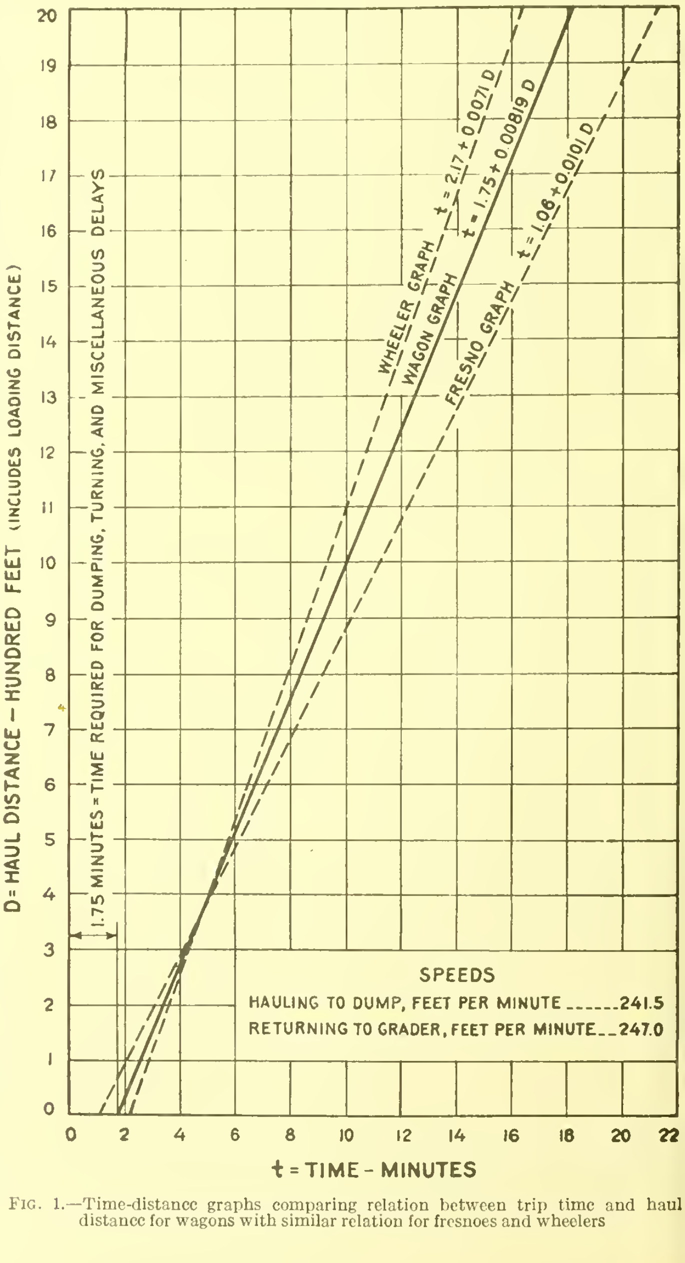

FIG. 1.—Time-distance graphs comparing relation between trip time and haul distance for wagons with similar relation for fresnoes and wheelers

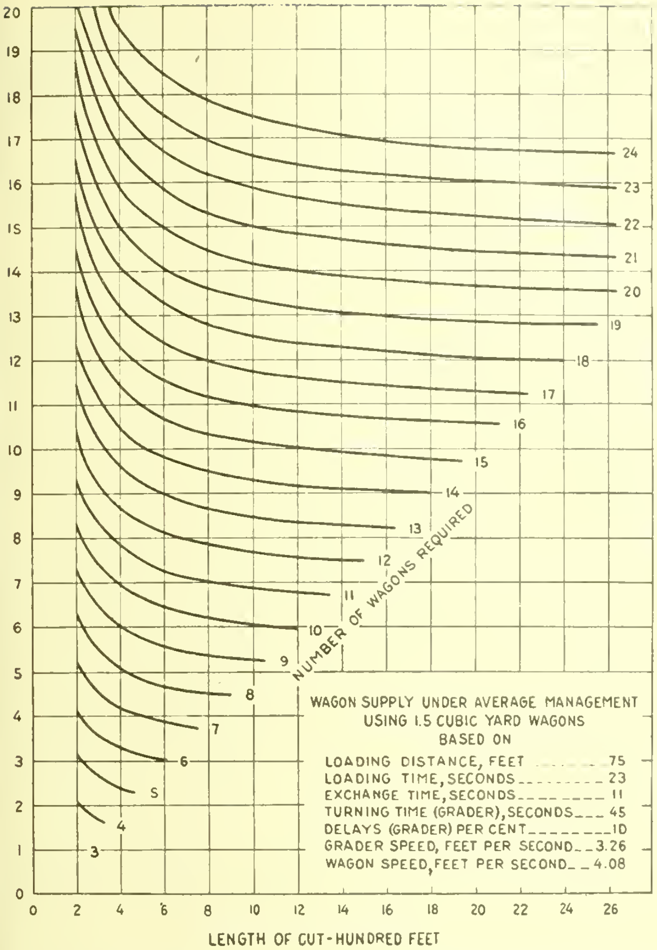

Fig. 2.—Curves for use in determining the supply of 1½ cubic-yard wagons required for various lengths of haul and lengths of cut with average management

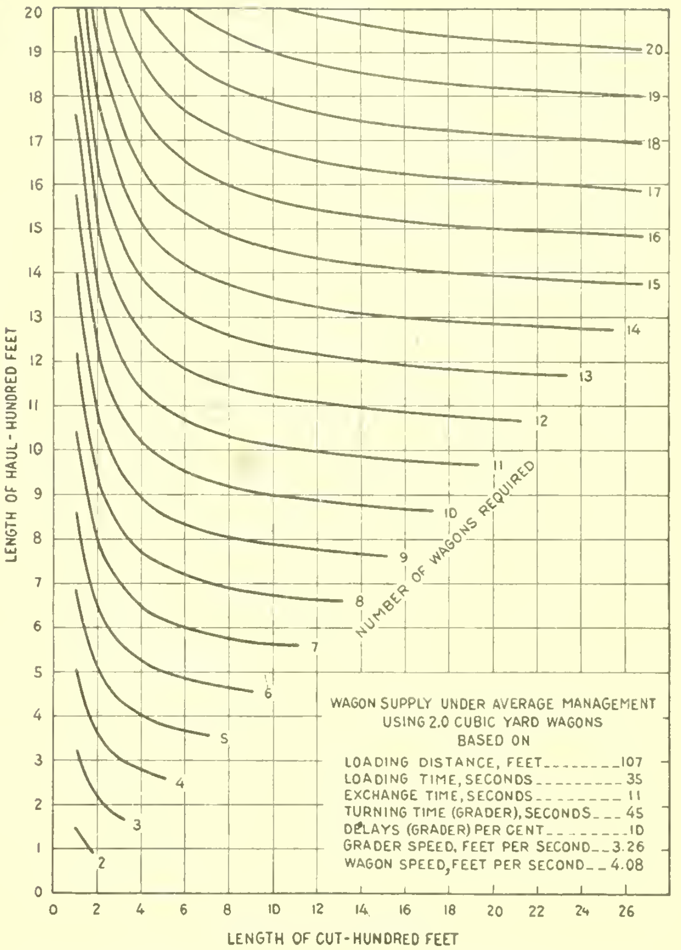

Fig. 3.—Curves for use in determining the supply of 2 cubic-yard wagons required for various lengths of haul and lengths of cut with average management

To demonstrate the use of the graphs, let us suppose the wagons used are of 1½ cubic yards capacity. We find from Figure 2 that under average management, with a length of cut of 600 feet and haul of 700 feet, the required wagon supply is 10.8 as previously stated. To determine the wagon supply needed under extra good management, we refer to Figure 4 and follow the vertical line for the 600-foot cut to its intersection with line B; then, following horizontally to the percentage scale at the left, we find that the wagon supply should be 160.5 per cent of the supply required for average management or (10.8×1.605) 17 wagons. For cuts of average length (approximately 450 feet long) the first five seconds saved (line X) increases the wagon supply required about 13 per cent, or at the rate of about 2.6 per cent for each second saved. As the increased wagon supply is required by increased production, the importance of each second saved is very apparent. The second five seconds saved (line Y) requires a further increase in wagon supply of about 17 per cent, or at the rate of 3.4 per cent per second. The third 5 seconds saved (line Z) requires an increased wagon supply at the rate of about 4.6 per cent per second. Where management is below the average the effect is, of course, reversed.

To use these graphs it is necessary to know what average loading time is being secured, the length of cut, the percentage of time the grader is idle, and the exchange time. If these conditions are as shown in Figure 2 or 3, which we call normal, the wagon supply is read directly from the graphs using Figure 2 or Figure 3, depending on whether 1½ or 2 cubic yard wagons are employed. If the observed conditions indicate that higher efficiency is being obtained determine the amount by which the normal loading time is reduced and, after ascertaining from Figure 2 or 3 the number of wagons required for average management, correct the result by increasing the number as indicated by the percentage read from Figure 3 or 4 opposite the intersection of the vertical line through the appropriate cut length figure with the horizontal line representing the time saved. Thus, by reference to Figure 2, we find that when working from a 450-foot cut on a 600-foot haul the wagon supply required under normal or average conditions is 9.2. But if a saving of 6 seconds is being made reference to Figure 4 shows that the wagon supply should be 116.4 per cent of the normal, or 11.

TIME LOSSES IN WAGON OPERATION

The constant losses involved in wagon operation are, in their general nature, similar to those encountered in the operation of other forms of earth-moving equipment. They include turning time, dumping, and miscellaneous waits. The field observations indicate that not much time is lost on the turns or in the dumping operation, which is simple and rapid. The principal avoidable losses are caused by: (1) Improper dump-

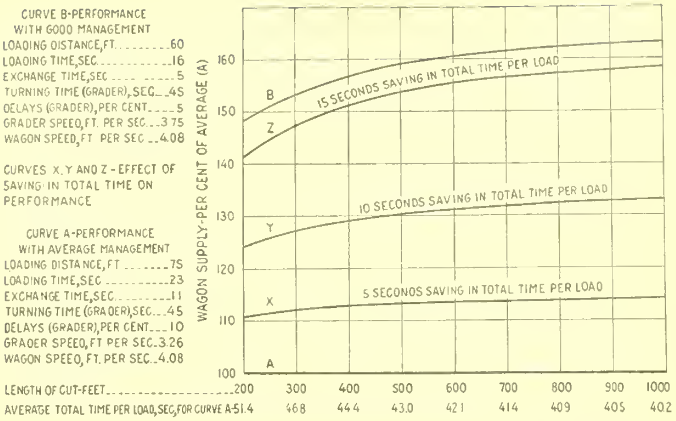

FIG. 4.—Increase in supply of 1½ cubic-yard wagons required by increased efficiency in operation of grader and wagons

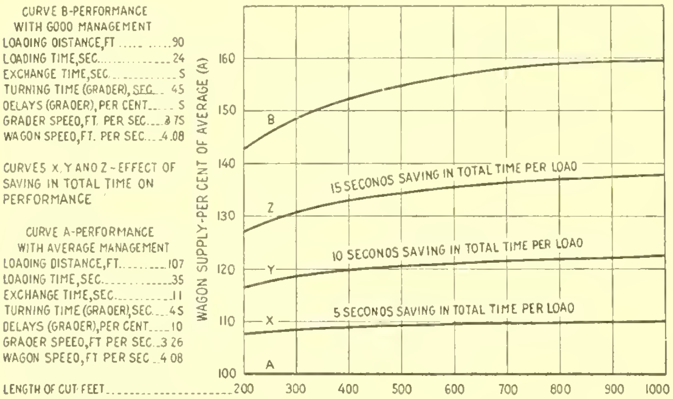

FIG. 5.—Increase in supply of 2 cubic-yard wagons required by increased efficiency in operation of grader and wagons

ing practices; (2) the practice, found on most jobs, of allowing teams to maintain a fixed position in the string; and (3) the use of an unnecessarily large number of wagons on the work. Loading (by the method of analysis used in this article) is included in the wagon travel time. A little time may be lost in loading by the checking of the normal wagon speed to conform to a lower grader speed, but this loss can be avoided only by a correction of the grader speed.

Handling material at the dump is an art. The dump man must have two separate objectives in mind—(1) the building of a neat and workmanlike fill and (2) the disposition of material as delivered without delaying the wagons. The first of these objectives is most easily accomplished by end dumping, for the reason that the dump man can preserve line and grade simply by setting a few shoulder stakes. But the practice of end dumping, aside from the fact that it produces a fill which will settle considerably more than a layer-dumped fill, is undesirable from the contractor's standpoint, because the dumping area is necessarily so small that the Mormon scraper or light blade used in distributing material and forcing it over the end of the dump can not work without more or less interruption by the wagons. Layer dumping enables the dump man to provide all the dumping area needed in handling material, no matter what the rate of delivery may be. On the other hand, it is a little more difficult to place a fill accurately if layer dumping is practiced and the number of dump men who can build a trim and regular layer fill is limited. That it can be done, however, with no additional cost to the contractor was demonstrated on a job studied at Jordan, Minn., during a considerable part of the past working season. The fills on this job, ranging from 1 or 2 feet to over 20 feet in depth, were placed by a skilled dump man and a two-horse blade with driver so accurately that even after allowing a reasonable period for settlement the engineers found that in many cases final dressing was not really required before the work could be accepted. This case is mentioned because it indicates the accuracy with which layer dumping can be performed if a really skilled dump man is in charge of the work.

On elevating-grader jobs it is customary to permit the drivers to maintain a regular position in the wagon string. Working under this system, it is not uncommon to see practically the whole string held up because one driver has stopped his team for some reason. It is even more common to see one or two wagons held up at the dump because one driver has been somewhat detained in dumping. In the course of a day's work, this practice may result in considerable delay. If the time losses at the dump are to be reduced as far as possible, it can best be accomplished by requiring teamsters to accept whatever position in the line falls to them. This would also measurably avoid one of the objections that some contractors have to layer dumping, namely, that it tends to disorganize the wagon string.

The last and perhaps the most important cause of miscellaneous delays is traceable to having too many wagons on the work. As now commonly operated the elevating-grader outfit consists of grader and a fixed number of wagons. These the contractor customarily sends out every morning without much regard to whether they can be used effectively or not. As a result, if the haul is short, there is a considerable surplus in the wagon supply which appears in long waits between loads, and sharply increases the lost wagon time. This has been discussed in another part of this article[1] and a method of avoiding the trouble suggested.

ESTIMATING THE COST OF ELEVATING-GRADER WORK

Turning now to the method of estimating, it may be stated at the outset that the process is somewhat complicated. In the first place, the contractor should know his general efficiency and whether or not he will expect as a matter of policy to supply an adequate wagon train. It has been made clear that where no particular attention is paid to the adequacy of the wagon train, the exchange of wagons at the grader is likely to reach 20 seconds or more as an average. If an adequate wagon supply is maintained, the exchange, under normal efficiency, drops to about 11 seconds; and, if the superintendence is of a high order, it can be driven down to about 5 seconds. Other phases of this matter might also be noted here, but will hardly be necessary in order to support the statement that any scheme of estimating, if successfully used, must reflect conditions of this sort. The procedure which is outlined below sets up a basis on which this can be done. As a method it may be followed with confidence, but it can not be followed blindly.

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[1] See Public Roads, April, 1925, p. 30.

Briefly stated, this method of estimating is based on the assumption that the output will run a little over 100 cubic yards an hour on a job where conditions are normal or average, i. e., an average length of cut in which to work the grader, average efficiency in operation, a normal bite, an adequate wagon supply, and normal time losses, factors which have been found to generate a 23-second average loading time, an 11-second period for the exchange of wagons, a 45-second grader turn, and (with a 16-horse grader) a time loss of about 10 per cent, due to breakdowns, clean outs, miscellaneous rests, etc. As managerial policies generally reduce the working day to something under 10 hours, the actual output, these conditions prevailing, should be about 1,000 cubic yards per working day. As estimating must be done from some general basis this one has been selected. The development of the estimate then becomes a matter of determining the probable working time on the basis of governing factors. It should be observed that the 1,000-cubic-yard basis is dependent on the assumption of average efficiency under average conditions. It is here that the contractor should inject known data as to his own efficiency. He may, for instance, regularly take a bite that is a little larger than average, or he may obtain a rate of wagon exchange a little better than average. These or other conditions may increase his output under standard conditions; or he may find that with his outfit he can not reach this rate. The point to be emphasized is that he should keep accurate records and in this way ascertain whether, with his particular outfit, he can reasonably use 1,000 cubic yards as a standard output or whether he should use 900 or perhaps 1,100 cubic yards.

In making up the estimate, the first factor to be considered is the length of cut. It has been shown in another part of this article[2] that the length of cut has a marked effect on the output which can be secured from the grader, Figure 6 shows the effect of length of cut when efficiency is normal for both 1½ and 2 cubic yard wagon outfits.

In determining the length of cut the mean length should be used. Thus, if a 3-foot cut is to be taken out in four levels which, by a study of the plans, are found to be 200, 340, and 420 feet long, the length of cut should be taken as the average or, say, 300 feet. This will be a little high if cuts are short, as the production falls off with disproportionate rapidity under 300 feet. However, the number of cuts requiring very short runs is generally so small that the error from this cause will not be great.

The first operation in preparing the estimate is, then, to determine the effect of the prevailing length of cut on the output, the yardage produced from a 450-foot cut being assumed to be 100 per cent. In the case of a 300-foot cut, the output of a 16-horse outfit, with normal efficiency, will be 93.4 per cent. (See fig. 6, graph A.) If the cut contains 5,400 cubic yards the equivalent yardage is, then, 5,400 cubic yards divided by 93.4 per cent or 5,800 yards; i. e., it will take as long to produce 5,400 cubic yards from a 300-foot cut as it would take to remove 5,800 cubic yards from a cut of average (450 feet) length. The equivalent yardage should be determined by 1,000 cubic-yard units and residual fractions as subsequent calculations are best handled in this way. The result of this determination for a specific job is shown in Table 3, the cut data being obtained from the profile shown in Figure 7 and the percentage of output from graph A in Figure 6.

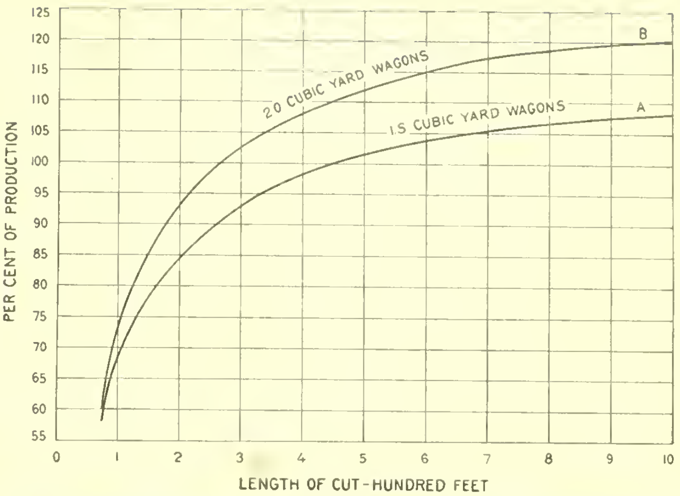

FIG. 6.—The effect of length of cut and size of wagons on production, operation with average management in a 450-foot cut with 1½ cubic-yard wagons being considered as 100 per cent. (See Table 2.)

The equivalent yardage would be a direct indication of the time required to handle the work if there were no long hauls. But there will generally be cuts from which the haul will be greater than can be handled by the wagon train that has been sent out with the grader. This will require a further adjustment of the equivalent yardage to allow for wagon shortage. In making this second adjustment the wagon travel is analyzed by the same 1,000-cubic-yard units to determine the wagon supply needed to handle each part of each cut; and, the needed supply being compared with the actual supply, another equivalent yardage is computed. The necessary wagon supply is read directly from the graphs shown in Figures 2 and 4 (or 3 and 5, depending on whether 1½ or 2 cubic-yard wagons are used); and the data on length of wagon travel and length of cut are secured from the mass diagram and the profile, repectively. The equivalent yardage due to length of cut and the equivalent yardage due to wagon supply are then scrutinized, and the controlling yardage is determined in each case by using the larger. The sum of the equivalent yardages so obtained divided by the average daily output (here assumed to be 1,000 cubic yards) is the number of days' work to be performed.

THE METHOD OF ESTIMATING ILLUSTRATED

For purposes of illustration fill 8 in Table 3 will be used. This fill contains 4,754 cubic yards, which must be obtained from a cut in which the average grader loop will be about 300 feet long. The average wagon travel, after adding 50 feet for wagon manipulation, is found from the mass diagram to be 1,175 feet for the first 1,000 cubic yards, 1,055 feet for the second 1,000 cubic yards, 940 feet for the third 1,000 cubic yards, 770 feet for the fourth, and 435 feet for the remaining 754 cubic yards of material. The percentage of production is found from Figure 6 to be 93.4 for a 300-foot cut. These data are tabulated in columns 2, 3, 4, and 5. Column 6, the result of dividing column 4 by column 5, gives the equivalent yardage due to length of cut and would be an indication of the time required to move the yardage into fill 8 if the wagon supply were adequate at all times. This wagon supply, as previously explained, is obtained from Figures 2 and 4 or 3 and 5, depending on whether 1½ or 2 cubic-

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[2] See Public Roads, May, 1925.

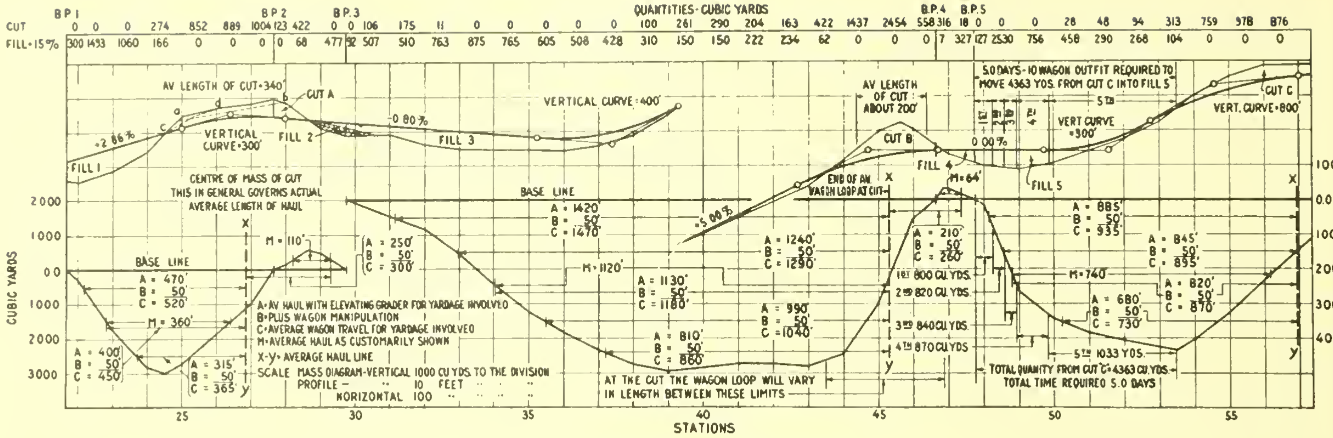
QUANTITIES-CUBIC YARDS
BP 1
BP 2
BP 3
B.P. 4
B.P. 5
CUT
FILL+15%
AV LENGTH OF CUT=340'
CUT A
VERTICAL CURVE=300'
FILL 1
FILL 2
FILL 3
+2.86%
-0.80%
VERTICAL CURVE=400'
CENTRE OF MASS OF CUT THIS IN GENERAL GOVERNS ACTUAL AVERAGE LENGTH OF HAUL
BASE LINE
CUBIC YARDS
A=AV HAUL WITH ELEVATING GRADER FOR YARDAGE INVOLVED
B=PLUS WAGON MANIPULATION
C=AVERAGE WAGON TRAVEL FOR YARDAGE INVOLVED
M=AVERAGE HAUL AS CUSTOMARILY SHOWN
X-Y=AVERAGE HAUL LINE
SCALE MASS DIAGRAM-VERTICAL 1000 CU YDS TO THE DIVISION
PROFILE – " 10 FEET
HORIZONTAL 100
M=360'
M=110'
M=1120'
AT THE CUT THE WAGON LOOP WILL VARY IN LENGTH BETWEEN THESE LIMITS
-5.00%
AV LENGTH OF CUT ABOUT 200'
CUT B
FILL 4
END OF AN WAGON LOOP AT CUT
M=64'
0.00%
FILL 5
5.0 DAYS-10 WAGON OUTFIT REQUIRED TO MOVE 4363 YDS FROM CUT C INTO FILL 5
VERT CURVE =300'
CUT C
VERT. CURVE=800'
1ST 800 CU.YDS.
2ND 820 CU.YDS.
3RD 840 CU.YDS.
4TH 870 CU.YDS.
5TH 1033 YDS.
M=740'
TOTAL QUANITY FROM CUT C=4363 CU.YDS.
TOTAL TIME REQUIRED 5.0 DAYS
STATIONS
25
30
35
40
45
50
55

Fig. 7.—Design of sample grading project

Table 3.—*Method of finding economical wagon supply. Cut data taken from design shown in Figure 7*

Fill No.	Length of cut	Haul, distance	Actual yardage	Percentage of production based on a 450-foot cut	Equivalent yardage due to length of cut	Wagon supply required	10-wagon outfit			11-wagon outfit			12-wagon outfit		
							Percentage of supply	Equivalent yardage due to wagon supply	Controlling equivalent yardage	Percentage of supply	Equivalent yardage due to wagon supply	Controlling equivalent yardage	Percentage of supply	Equivalent yardage due to wagon supply	Controlling equivalent yardage
1	2	3	4	5	6	7	8	9	10	8	9	10	8	9	10
	Feet	*Feet*	*Cubic yards*	*Per cent*	*Cubic yards*	*Number*	*Per cent*	*Cubic yards*	*Cubic yards*	*Per cent*	*Cubic yards*	*Cubic yards*	*Per cent*	*Cubic yards*	*Cubic yards*
1.	340	520	1,000	95.5	1,048	8	100	1,000	1,048	100	1,000	1,048	100	1,000	1,048
		450	1,000	95.5	1,048	7	100	1,000	1,048	100	1,000	1,048	100	1,000	1,048
		365	1,019	95.5	1,067	6	100	1,019	1,067	100	1,019	1,067	100	1,019	1,067
2.	340	300	545	95.5	571	6	100	545	571	100	545	571	100	545	571
3.	200	1,470	1,000	84.1	1,190	16	63	1,585	1,585	69	1,450	1,450	75	1,335	1,335
		1,290	1,000	84.1	1,190	15	67	1,495	1,495	73	1,370	1,370	80	1,250	1,250
		1,180	1,000	84.1	1,190	13	77	1,300	1,300	85	1,175	1,190	93	1,075	1,190
		1,040	1,000	84.1	1,190	12	83	1,205	1,205	92	1,090	1,190	100	1,000	1,190
		860	725	84.1	863	10	100	725	863	100	725	863	100	725	863
		350	300	84.1	357	5	100	300	357	100	300	357	100	300	357
4.	200	260	315	84.1	375	4	100	315	375	100	315	375	100	315	375
5.	550	935	1,000	102.5	976	13	77	1,300	1,300	85	1,175	1,175	93	1,075	1,075
		895	1,000	102.5	976	13	77	1,300	1,300	85	1,175	1,175	93	1,075	1,075
		870	1,000	102.5	976	13	77	1,300	1,300	85	1,175	1,175	93	1,075	1,075
		730	1,000	102.5	976	11	91	1,100	1,100	100	1,000	976	100	1,000	976
		500	340	102.5	332	8	100	340	332	100	340	332	100	340	332
6.	550	700	250	102.5	244	11	91	275	275	100	250	244	100	250	244
6a.	550	1,765	1,000	102.5	976	23	43	2,330	2,330	48	2,085	2,085	52	1,925	1,925
		1,720	180	102.5	176	22	45	400	400	50	360	360	55	327	327
	300	665	820	93.4	878	9	100	820	878	100	820	878	100	820	878
		600	1,000	93.4	1,070	8	100	1,000	1,070	100	1,000	1,070	100	1,000	1,070
		510	1,000	93.4	1,070	8	100	1,000	1,070	100	1,000	1,070	100	1,000	1,070
		415	1,000	93.4	1,070	7	100	1,000	1,070	100	1,000	1,070	100	1,000	1,070
		325	340	93.4	364	6	100	340	364	100	340	364	100	340	364
7.	300	465	1,000	93.4	1,070	7	100	1,000	1,070	100	1,000	1,070	100	1,000	1,070
		335	252	93.4	270	6	100	252	270	100	252	270	100	252	270
8.	300	1,175	1,000	93.4	1,070	15	67	1,495	1,495	73	1,370	1,370	80	1,250	1,250
		1,055	1,000	93.4	1,070	13	77	1,300	1,300	85	1,175	1,175	93	1,075	1,075
		940	1,000	93.4	1,070	12	83	1,205	1,205	92	1,090	1,090	100	1,000	1,070
		770	1,000	93.4	1,070	10	100	1,000	1,070	100	1,000	1,070	100	1,000	1,070
		435	754	93.4	808	7	100	754	808	100	754	808	100	754	808
Casting			2,256	100.0	2,256				2,256			2,256			2,256
Total yardage			27,096						33,177			31,612			30,644

Cost per cubic yard for 10 wagons (outfit cost $130 per day) 15.9 cents, for 11 wagons (outfit cost $135 per day) 15.7 cents, for 12 wagons (outfit cost $140 per day) 15.8 cents.

yard wagons are used; and in this illustration is found to range from 7 to 15 wagons (Figure 2 because average management has been assumed).

From the previous discussion and from column 7 of Table 3 it can readily be seen that an adequate wagon supply at all times would not only be uneconomical but practically impossible. Therefore, an economic wagon supply must be determined and for the first trial a 10-wagon outfit will be assumed. The first step then is to find the percentage relation of this trial outfit to the full wagon supply theoretically required to move each 1,000-cubic-yard unit

$$\text{column } 8 = \frac{\text{size of trial outfit}}{\text{column } 7}.$$

For a 10-wagon outfit, this relation for each of the units moved to fill 8 is $\frac{10}{15}, \frac{10}{13}, \frac{10}{12}, \frac{10}{10}$, and $\frac{10}{7}$, or 67, 77, 83, 100, and 100 per cent, respectively. It will be noted that where the number of wagons tested exceeds the number of wagons required, the percentage of wagon supply is still 100 per cent. This is due to the fact that the elevating grader has a definite capacity and, when the wagon supply exceeds the required supply production does not increase; the wagons simply lose time waiting for their loads. Now, by dividing the actual yardage by the percentage of wagon supply (column 4 divided by column 8) we obtain the equivalent yardage due to wagon supply (column 9). We

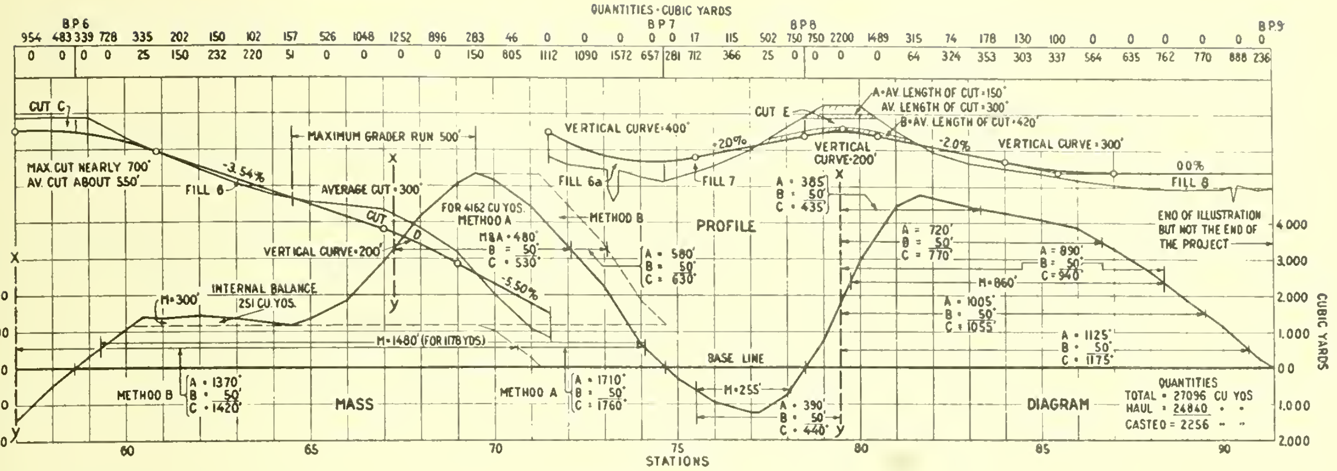

FIG. 7—Continued

compare this yardage with the equivalent yardage due to length of cut (column 6) and in each case place the larger yardage, which is the controlling equivalent yardage, in column 10. Material cast directly from the grader to the subgrade should also be tabulated in column 10. The sum of the controlling equivalent yardages (column 10) divided by the average daily output (assumed as 1,000 cubic yards) gives the number of days required to place the actual yardage shown in column 4. Similarly the time required for the movement of the same material with 11 and 12 wagon outfits may be determined; and it will be found that 33.2 days will be required for a 10-wagon outfit, 31.6 days for an 11-wagon outfit, and 30.6 days for a 12-wagon outfit.

The field costs will amount to about $130, $135, and $140 per day, respectively (based on $80 per day for the grader unit and $5 per day for each wagon).[3] Therefore, the unit cost of production on this job with a 10-wagon outfit will be $\frac{33.2\times 130}{27,096}=15.9$ cents per cubic yard; for an 11-wagon outfit, $\frac{31.6\times 135}{27,096}=15.7$ cents per cubic yard; and for a 12-wagon outfit, $\frac{30.6\times 140}{27,096}=15.8$ cents per cubic yard, indicating that the economical supply is 11 wagons.

There are some minor adjustments which could be made in this sytsem of estimating to slightly increase its accuracy. The method used, however, has been found to yield results generally differing from those of the more detailed methods by less than a tenth of a cent per cubic yard. These minor adjustments, therefore, are felt to be of no practical value.

THE EFFECT OF DESIGN ON COST OF GRADING

There is a great variation in the relation between equivalent yardage and the actual yardage from project to project. This difference brings into view the effect of the elements of risk which have been discussed in this series of articles and offers at once a convenient and a valuable basis from which to study both design and the cost of construction. Obviously, the ideal project would be one so designed that the grader could always produce at least a standard output and that a normal wagon train could at all times handle all of this output. In that case actual yardage would equal equivalent yardage. But projects so designed are rare. Table 4 shows the actual yardage, the equivalent yardage, and the relation between them, for various wagon trains which might be selected for use on the three designs, A, B, and C, discussed in the second part of this article.[4]

In the original design, A, with the most economical length of wagon train, equivalent yardage exceeds actual yardage by about 17 per cent. This means, of course, that production will cost 17 per cent more on this project than it would cost on a project of ideal design requiring the same wagon train. In the first redesign, with the most economical length of wagon train, equivalent yardage exceeds actual yardage by about 13 per cent, and in the second redesign the excess is about 11 per cent. This indicates that in each of these designs there has been a failure to completely adjust the design to the most economical length of wagon train. From the contractor's standpoint it means that average production will unavoidably be below standard.

Such an analysis as the one here developed also makes it possible to select the most economical wagon train.

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[3] Approximate field costs:

Grader:	
17 horses (16 regular, 1 extra)	$12.00
1 eight-horse driver (high seat)	6.00
2 four-horse drivers	9.00
1 plow shaker	10.00
1 wheel-stock man	4.50
	41.50
Dump:	
1 two-horse team	1.50
1 driver	3.50
1 dump man	8.50
	13.50
Miscellaneous:	
1 blacksmith	7.50
1 foreman	11.50
1 laborer (miscellaneous work)	3.00
Miscellaneous light repairs	3.00
	25.00
Wagons:	
10 wagons at $5.00	50.00
11 wagons at $5.00	55.00
12 wagons at $5.00	60.00
Total field costs:	
10-wagon outfit	130.00
11-wagon outfit	135.00
12-wagon outfit	140.00

The above figures are illustrative only. In a general way they reflect field costs as they are to-day, but there is so much difference in wage standards in different camps, particularly in the higher positions, as well as in the composition of the working force—that no estimate of this sort can be set up as standard. Each contractor must make his own estimate based on his own wage scale and organization.

[4] See PUBLIC ROADS, vol. 6, No. 3, May, 1925.

Table 4.—*Actual and equivalent yardage and their relationship for designs A, B, and C*

Design	6-wagon outfit			7-wagon outfit			8-wagon outfit			9-wagon outfit			10-wagon outfit			11-wagon outfit			12-wagon outfit		
	Actual yardage	Equivalent yardage	Increase	Actual yardage	Equivalent yardage	Increase	Actual yardage	Equivalent yardage	Increase	Actual yardage	Equivalent yardage	Increase	Actual yardage	Equivalent yardage	Increase	Actual yardage	Equivalent yardage	Increase	Actual yardage	Equivalent yardage	Increase
	Cubic yards	*Cubic yards*	*Per cent*	*Cubic yards*	*Cubic yards*	*Per cent*	*Cubic yards*	*Cubic yards*	*Per cent*	*Cubic yards*	*Cubic yards*	*Per cent*	*Cubic yards*	*Cubic yards*	*Per cent*	*Cubic yards*	*Cubic yards*	*Per cent*	*Cubic yards*	*Cubic yards*	*Per cent*
Original, A	------	------	------	------	------	------	------	------	------	------	------	------	27,096	33,177	22	27,096	[1] 31,612	17	27,096	30,644	13
First redesign- B	------	------	------	------	------	------	26,573	31,480	19	26,573	[1] 29,887	13	26,573	28,947	9	------	------	------	------	------	------
Second redesign, C	27,446	33,663	23	27,446	[1] 30,445	11	27,446	29,419	7	------	------	------	------	------	------	------	------	------	------	------	------

[1] Economical wagon train.

It will be observed that the number of wagons assigned to an outfit may materially influence the unit cost. This is, of course, due to the fact that as the size of the outfit increases the percentage of the total material which can be hauled without delaying the grader increases. However, the cost of operating the outfit increases as the wagon train increases. The two, therefore, tend to offset each other, a circumstance which accounts for the fact that for each job there is an outfit most economical for the haul conditions prevailing. So important is this latter phase that it should always be examined quite as carefully as the effective yardage. Thus, in the designs which have been used as an illustration, the most economical wagon train for design A is 11 wagons; for design B, 9 wagons; and for design C, 7 wagons. Increases or decreases from these wagon supplies would tend to cause increased production cost. This also has a bearing on cost, for obviously an outfit operating 11 wagons will cost more than one operating 7 wagons. In this case the net result is that under the original design even when 11 wagons are used the work will take 17 per cent more time than an ideal design would require, while in case C, with 7 wagons working, about 11 per cent excess time is required. The effect on production cost is obvious.

With an estimate developed in this way the contractor may schedule his work as described in the previous articles on wheelers and fresnoes.[5] It is not necessary to redescribe the method here. It may, however, be remarked that a time schedule of operation is quite as valuable a control on this work as on other grading work.

THE EFFECT OF THE CHARACTER OF THE EARTH MOVED

Nothing has been said in this article in regard to differences in the character of the material to be handled as affecting cost. The reason for this is that, as with other forms of dirt-moving equipment, the character of the material, while often differing a good deal from cut to cut, tends to average into rather a constant factor. Undoubtedly there is a difference in the power required in taking out different classes of material. In some the bull wheel functions better than in others. The dynamometer studies, now in progress, are expected to yield data on these points. In rare instances, as where fine dry sand, especially heavy soil, and ground tending to become marshy, etc., will be encountered in considerable amount, the contractor should make allowance for soil conditions by reducing the assumed output from 10 to 30 or perhaps 40 per cent, as his experience may dictate. This will not, however, correspondingly increase production cost as his wagon train can be somewhat reduced if the output is likely to be consistently low.

Where bowlders will be encountered, the cost of the outfit per day should be modified to include under miscellaneous expenses the men and teams needed to get the rocks out of the working area. On the projects which have been studied the bowlders have varied from none to enough to keep two teams, with drivers and two extra men, at work all the time. As bowlders, even in some quantity, do not materially affect the output of the grader, the expense of removing them should be estimated as a miscellaneous expense, but generally there need be no allowance for reduction in output.

There are a number of other aspects of elevating-grader work which deserve consideration. Some of these require further study and others can be more appropriately considered in a general discussion of grading work. They are therefore omitted at this time.

WENDOVER CUT-OFF COMPLETED

The official opening of the Wendover Cut-off across the mud flats and salt beds of the Great Salt Lake Desert was held on June 13 at Salduro, Utah. The completion of the road, 41 miles long, opens the way for transcontinental and interstate travel between Salt Lake City and northern and central California and brings to a successful conclusion a five-year effort to bridge the age-old obstacle of the salt desert.

A part of the road is built upon a solid layer of salt, which is permanently protected from disintegration by a clay cut-off wall. In the construction work horses could be employed only with the greatest difficulty; all fresh water had to be hauled for many miles, and for several miles the roadbed was completely submerged in brine for months at a time. Excavating machinery of a new design was necessary to handle the material from beneath the salt beds to its place in the embankment.

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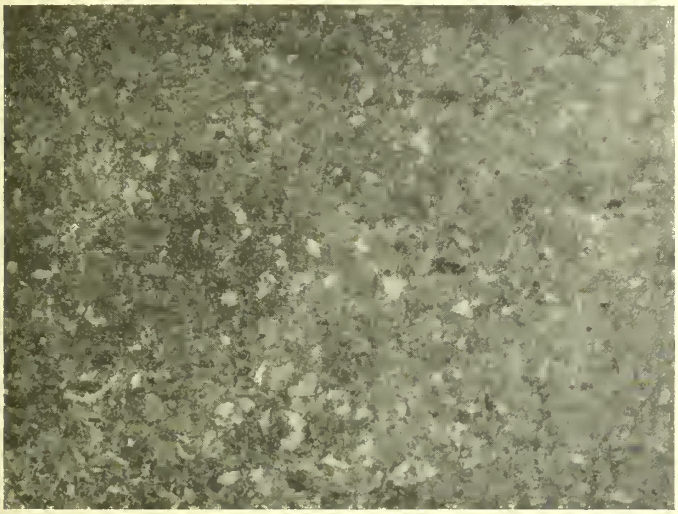
[5] See Public Roads, vol. 5, No. 7, September, 1924; vol. 5, No. 8, October, 1924; and vol. 5, No. 10, December, 1924.

CONDITION OF THE OHIO POST ROAD AFTER 10 YEARS UNDER TRAFFIC

A SURVEY MADE FOR THE DIVISION OF TESTS, UNITED STATES BUREAU OF PUBLIC ROADS

Reported by **F. H. JACKSON**, Engineer of Tests

ONE OF the first concrete pavements of any considerable length to be constructed in the United States was laid in 1914 and 1915 on a section of the old National Road between Zanesville, Ohio, and the Moscow bridge over the South Fork of the Licking River, a distance of about 24 miles. The pavement, built under the joint supervision of the State of Ohio and the United States Bureau of Public Roads and known as the Ohio Post Road, has now been under traffic for more than 10 years and is still in good condition.


The present condition of the Ohio Post Road is generally good. There is no scaling and only a moderate amount of surface wear

Fortunately there is available a detailed record of the materials and methods used in the construction and there is the possibility, so often lacking, of studying the present condition in the light of the conditions obtaining during construction. With this possibility in view the road was inspected for the Bureau of Public Roads in November, 1924, and observations made at that time are recorded in this article.

One condition that stands out very clearly is the greater amount of cracking that has occurred in parts of the pavement in which uncrushed gravel was used as the coarse aggregate in comparison with other parts in which the aggregate was crushed limestone. There were about twice as many full transverse cracks per mile in the gravel as in the limestone concrete sections.

Another noticeable condition is the entire absence of scaling, which the writer attributes to the fact, as evidenced by the records, that the sand was exceptionally clean, the mix dry and the finishing moderate. So excellent is the surface in this respect, and so strong the indications of the records that the result is due to the construction conditions mentioned, that the writer feels justified in saying that more careful attention to these details should go far toward eliminating scaling on new construction.

The observations also add to the evidence which is piling up from similar investigations in many parts of the country that there is an intimate relation between the extent of cracking in concrete pavements and the character of the soil immediately below the pavement. In the Ohio Post Road, as in the Columbia Pike in Virginia,[1] and other roads which have been the subject of recent reports, the greatest amount of cracking has occurred on soils of high clay content.

GENERAL DESCRIPTION OF THE ROADS

Ten of the 24 miles were surfaced with 1: 1½: 3 concrete using the uncrushed gravel aggregate, the remaining 14 miles with 1: 1¾: 3 concrete in which the aggregate was crushed limestone. Throughout its entire length the pavement is 16 feet wide and the slab, which is 8 inches thick at the center and 6 inches at the edges, rests on a flat subgrade. Contraction joints were spaced 30 feet apart and were skewed at an angle of 15° with the center line of the road. One thickness of two-ply tar paper was used in the joints except for a few miles on which ¼-inch premolded expansion joints were tried.

The macadam surface over which the concrete was laid had received practically no attention for a number of years prior to the improvement so that it was in a very poor state of repair. In many places the stone had disappeared entirely and during wet seasons, sections of the road became almost impassable. With few exceptions the line of the survey for the improvement followed the center line of the old road, and in no case was the departure greater than 10 or 15 feet. The maximum grade of the improved road is 7 per cent. The soil of the region which the road traverses consists for the most part of a heavy red clay with here and there an admixture of blue pipe clay which is very difficult to drain. The topography is rather rugged, affording good natural drainage for the most part, there being only about 1¾ miles at the extreme western end of the improvement—a practically level stretch—in which this condition does not obtain.

Pavement in excellent condition. Longitudinal cracks show a negligiolc amount of vertical faulting

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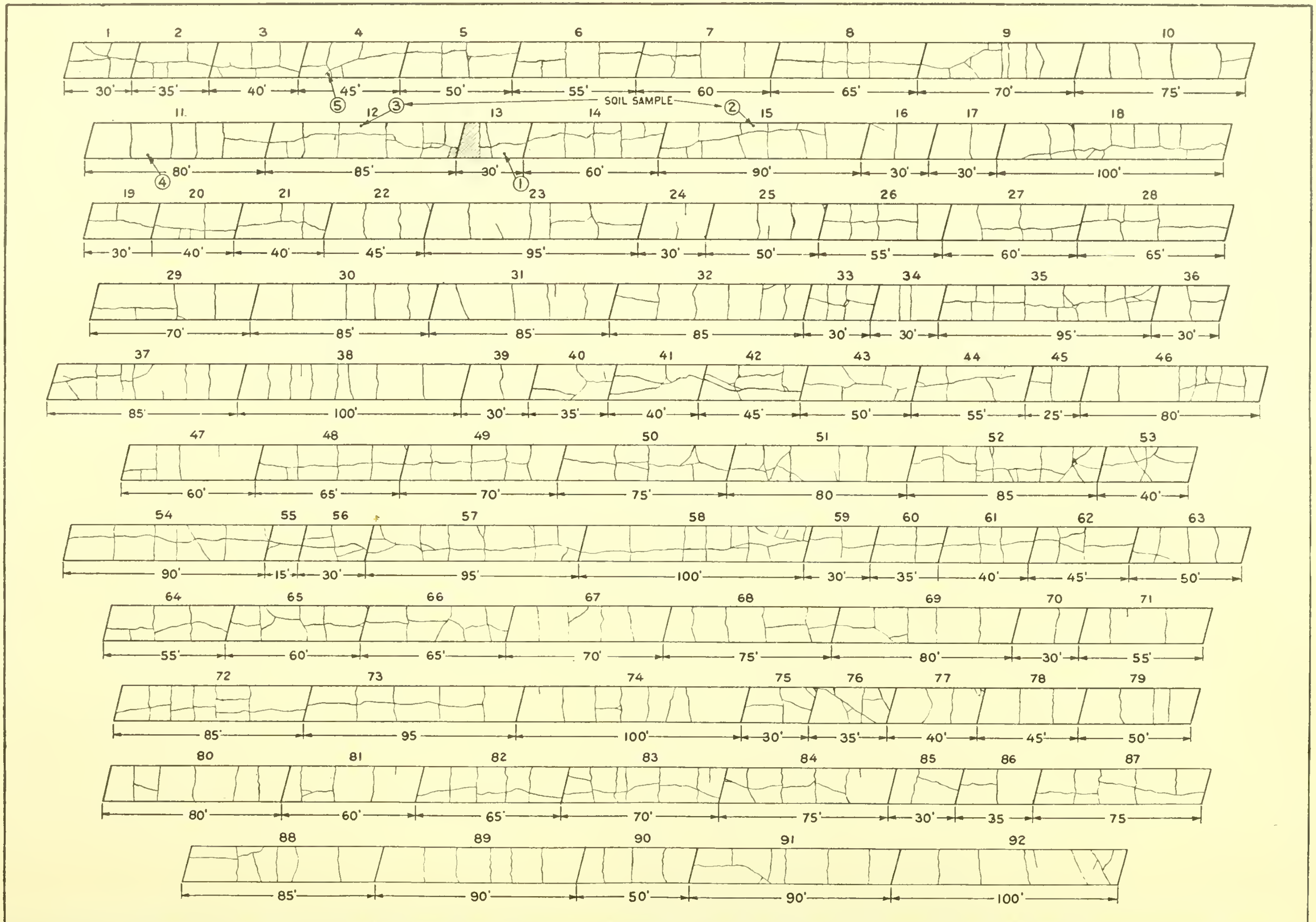
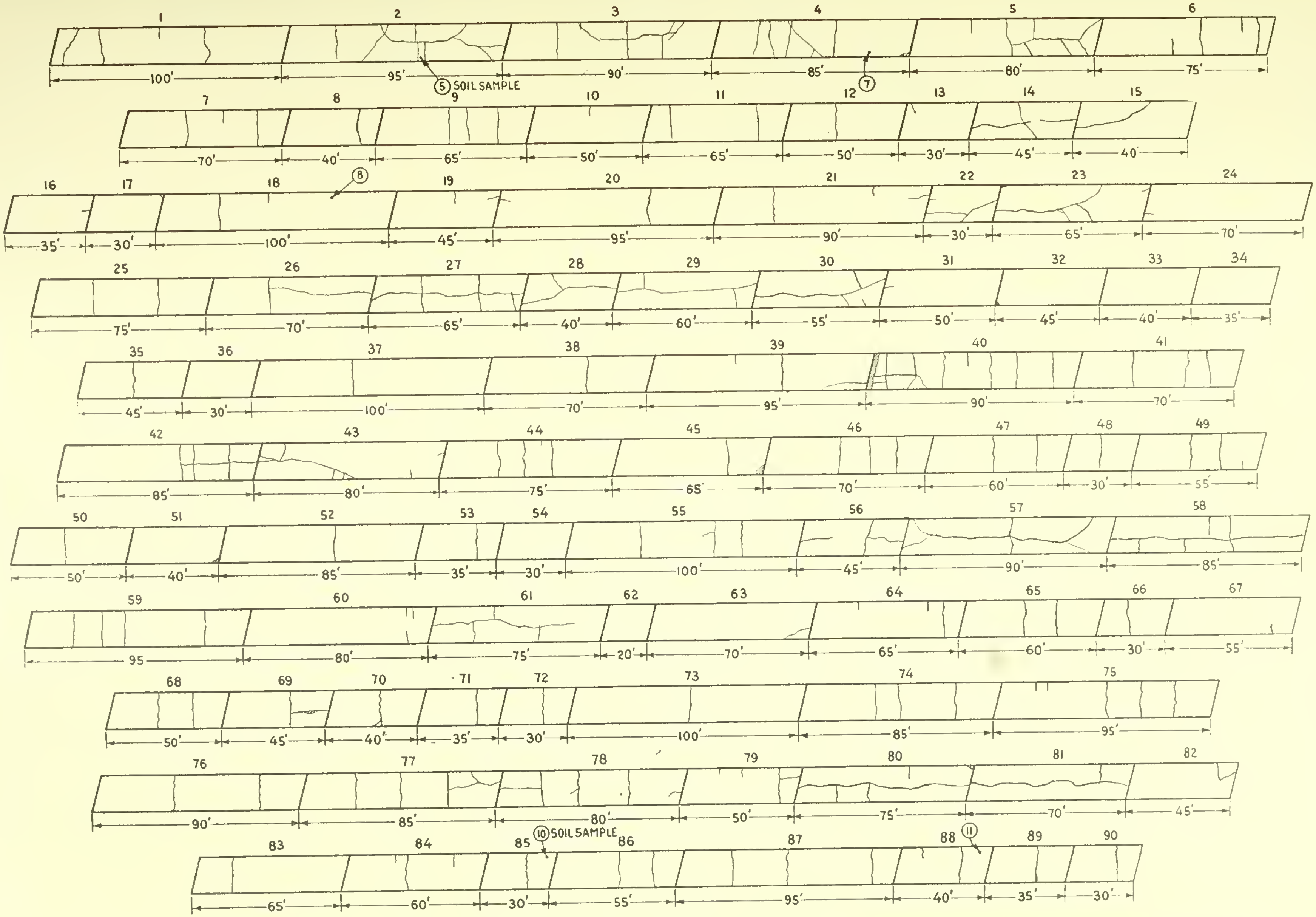
[1] See PUBLIC ROADS, vol. 5, No. 8, October, 1924.



NOTE:- FIGURES ABOVE AND AT CENTER OF EACH SECTION INDICATE NUMBER OF SECTION. FIGURES BELOW AND AT CENTER OF EACH SECTION INDICATE LENGTH OF SECTION IN FEET. SHADED AREAS INDICATE BITUMINOUS CONCRETE REPLACEMENT

FIG. 1.—Crack survey of experimental section of Ohio Post Road in Muskingum County. This section was surfaced with 1:1½:3 gravel concrete. The cracks noted were present in the pavement on Nov. 10, 1924.



NOTE – FIGURES ABOVE AND AT CENTER OF EACH SECTION INDICATE NUMBER OF SECTION. FIGURES BELOW AND AT CENTER OF EACH SECTION INDICATE LENGTH OF SECTION IN FEET. SHADED AREAS INDICATE BITUMINOUS CONCRETE REPLACEMENT

FIG. 2.—Crack survey of experimental section of Ohio Post Road in Licking County. This section was surfaced with 1:1¾:3 limestone concrete. The cracks noted were present in the pavement on Nov. 10, 1924.

MATERIALS AND CONSTRUCTION METHODS

Records of the physical tests made on the Portland cement used indicate that it passed all of the usual requirements.

The sand, obtained from a washing plant at Dresden, Ohio, was exceptionally well graded and clean, as indicated by the results of the typical analyses and tests shown in Table 1. In view of the following description of the surface appearance of the pavement to-day, it is of interest to note that the maximum silt content shown by any of these analyses was 2.6 per cent, and that the maximum passing a 100-mesh sieve was 4 per cent. In only one of the eight analyses reported was the silt content greater than 1.3 per cent.

As previously stated, both uncrushed gravel and crushed limestone were used as coarse aggregates. The gravel was obtained from the same source as the sand and was of good quality. The limestone, also of excellent quality, was obtained from one of the Scioto River quarries near Columbus, with the exception of that used in about 4,500 feet of the pavement, which was obtained from a local quarry. The characteristics of both materials are indicated in Table 2.

The water, pumped from streams crossed by the road, was also of good quality.

TABLE 1.—*Results of tests of sand*

Sample No.	Mechanical analysis—Total retained on sieve number							Silt
	10	20	30	40	50	80	100	
	Per ct.	*Per ct.*	*Per ct.*	*Per ct.*	*Per ct.*	*Per ct.*	*Per ct.*	*Per ct.*
1	18	34	49	63	84	97	98	0.7
2	16	33	52	69	91	99	99	0.3
3	8	21	44	64	88	97	98	1.3
4	14	26	47	62	81	93	96	0.5
5	10	22	42	60	88	99	99	0.0
6	4	19	45	68	89	96	98	0.5
7	17	41	65	78	91	96	98	1.3
8	19	41	62	74	88	95	97	2.6

TABLE 2.—*Results of tests of coarse aggregate*

Sample No.	Material	Mechanical analysis—total retained on screens					Wear	Toughness	Hardness
		1½-inch	1-inch	¾-inch	½-inch	¼-inch			
		Per ct.	*Per ct.*	*Per ct.*	*Per ct.*	*Per ct.*	*Per ct.*	*Per ct.*	*Per ct.*
1	Limestone.	9	44	73	96	100			
2	do	3	59	80	97	100			
3	do		13	51	90	100			
4	do						3.4	8	16.7
5	do						3.2	5	15.8
6	do						3.3	12	18.5
7	do						3.9	15	17.6
8	Gravel		5	31	72	92			
9	do		24	45	73	96			
10	do		4	15	37	82			
11	do	1	6	17	49	96			
12	do		6	17	50	86			

All materials were conveyed from the railroad sidings to the job by means of an industrial railway constructed along the right of way. Aggregates were stored on the subgrade and were proportioned by means of wheelbarrows of such size that the desired quantities were secured by striking off the materials level with the tops. The concrete was mixed in one-half cubic yard paving mixers of the boom-and-bucket type, and in general the mixers were so operated that each batch of concrete remained in the drum at least 45 seconds. Great care was exercised to regulate the consistency of the mix so that the concrete would hold its shape after being placed, struck off, and finished. All reports on file indicate that more than usual attention was given to this feature, and that the concrete was, if anything, too dry rather than too wet. This point is also of interest in view of the present condition of the pavement. The concrete was struck off by means of a wooden strike board and tamped with a steel templet 6 inches wide and shaped to conform to the surface.

The work of finishing the surface proceeded immediately after the tamping and was accomplished by means of wooden floats of two sizes; one about 8 feet long, operated by two men, and the other about 2 feet long, operated by one man. The long float was operated from two bridges and was shoved back and forth longitudinally across the pavement. The short one was for final finishing. For the most part the surface was brought to final finish not more than 50 feet behind the mixer. Although the finish was good, there was no attempt to secure a surface of exceptional smoothness, such as is frequently made in current practice. There was consequently little tendency toward overfinishing. Thus, it is noted that three of the factors which from time to time have been brought forward as causing surface scaling, viz, excessive fine material in the sand, wet mix, and overfinishing, were conspicuously absent on this job.

A canvas cover was spread over the surface of the concrete as soon after it was finished as possible. From 15 to 24 hours after laying, the canvas was removed and a 2-inch earth covering was placed and kept continuously wet for a period of two weeks, after which it was removed and the road opened to traffic.

Typical results of compressive tests of concrete cylinders taken from time to time during the progress of the work are shown in Table 3. It will be noted that the crushing strength at 14 days of the 6 by 6 inch cylinders of both kinds of concrete ranged from about 2,400 to about 3,400 pounds per square inch. The average strength of both the gravel and the limestone concrete at 14 days was approximately 2,800 pounds per square inch.

TABLE 3.—*Crushing strength of stone and gravel concrete*

Specimens: 6 by 6 inch cylinders.
Age: 14 days (10 days under damp earth).

Set	Crushing strength of 1:1½:3 gravel concrete	Crushing strength of 1:1¾:3 limestone concrete	Set	Crushing strength of 1:1½:3 gravel concrete	Crushing strength of 1:1¾:3 limestone concrete
	Pounds per square inch	*Pounds per square inch*		*Pounds per square inch*	*Pounds per square inch*
1	3,280	3,430	5	2,855	2,935
	3,170	3,060		2,745	3,210
		3,840			3,250
2	2,440	2,660	6	2,320	[1] 3,070
	2,450	2,770		2,650	[1] 3,180
	2,520	2,760		2,820	[1] 3,430
3	2,395	2,415	7	2,760	[1] 2,790
	2,550	2,335		2,995	[1] 3,160
	2,270	2,390		2,925	[1] 3,275
4	3,350	2,390	8	3,120	
	3,120	2,440		2,620	
	3,620			3,490	

[1] Specimens 28 days old and 14 days under damp earth.

RELATIVE CRACKING OF GRAVEL AND LIMESTONE CONCRETE

As the same sand and cement and the same details of construction were used throughout, the possibility of comparing the behavior of the two kinds of concrete was anticipated when the road was built, and two special experimental sections were laid. The sections were so located as to have practically identical traffic, subsoil, and drainage conditions. Both were constructed through gently rolling country; each contained approximately the same amount of pavement through cut and on fill; and each was constructed with contraction joints varying from 30 to 100 feet apart, the distance changing by increments of 5 feet and repeating for a distance of approximately 1 mile in each case. All the conditions on these two experimental sections being so nearly alike with the exception of the kind of coarse aggregate it is interesting to compare the relative amount of transverse cracking which has taken place in them during the 10 years since they were constructed, and with this in mind a detailed crack survey was made at the time of the recent inspection.

The results of the crack survey are reproduced in Figures 1 and 2. It will be seen at once that many more cracks appear in the uncrushed gravel section than on the crushed limestone section. The relative amount of cracking may be determined in a number of ways. For instance, by counting the actual number of concrete slabs formed by the cracks and joints on each section and dividing into the total area of the section, we obtain the average area of concrete slab in each section after the 10 years of service. Calculated in this way, we find that the average area of the concrete slabs as they exist to-day on the gravel section is 137 square feet, whereas the average area of the slabs on the limestone section is 320 square feet—about two and one-half times greater. Comparison of the two sections likewise shows that, whereas on the gravel section every one of the original slabs has cracked at least once, on the limestone section there are 11 slabs still intact. One of these is 20 feet long; four are 30 feet long, two, 35 feet long; two, 40 feet long; one, 45 feet long; and one, 50 feet long. There are 230 full transverse cracks on the gravel section and 107 on the limestone section. The full transverse or contraction cracks on the gravel concrete section average 24 feet apart as against 54 feet on the limestone concrete section.

Inasmuch as soil, drainage, and traffic conditions are practically identical, it would appear that the difference in cracking was due to some characteristic of the concrete itself. As previously stated, a slightly richer mix was used in the gravel concrete—i. e., 1:1½:3 as against 1:1¾:3 for the limestone concrete—but all other factors remained exactly the same, with the exception of the coarse aggregate. It is fair to assume, therefore, that the difference in the cracking of these two sections is due to the character of the coarse aggregate used. Results of typical compression tests of the concrete made on specimens cast at the time of construction are given in Table 3. It will be noted that there is very little difference in the crushing strength of the two classes of concrete. Unfortunately, there are no transverse tests of the concrete available, but it is possible that the difference in cracking is due to the high tensile strength of the limestone concrete.

RELATION OF CRACKING TO CHARACTER OF SUBGRADE SOIL

During the course of the inspection, 10 samples of subgrade soil were obtained from beneath the edges of the pavement—5 on the crushed limestone section and 5 on the uncrushed gravel section. Certain of these samples were taken at points where extensive cracking had taken place and others on sections which had cracked very little. The results of physical tests of the samples are given in Table 4, and the exact locations from which they were taken are indicated on Figures 1 and 2. The figures in circles on these charts indicate the numbers of the soil samples.

TABLE 4.—*Results of physical tests of subgrade soils*

Sample No.	Relative amount of cracking in concrete [1]	Classification of soil	Mechanical analysis				Moisture equivalent	Capillary moisture	Volumetric shrinkage	Dye adsorption No.
			Material in original sample retained on 10-mesh sieve	Material passing 10-mesh sieve						
				Sand	Silt	Clay				
			P. ct.	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	
1	Large	Clayey silt	5	15	43	42	20	30	17	23
2	do	Silty clay	9	11	42	47	19	29	15	19
3	do	do	8	8	26	65	22	31	23	26
4	Small	Clayey sand	18	45	19	36	15	29	20	16
5	Large	Silty clay	9	12	34	54	21	29	16	24
6	do	do	6	18	35	47	22	29	22	38
7	Small	do	7	28	35	37	18	26	18	14
8	do	do	15	16	31	53	23	37	18	23
10	do	Clayey silt	17	29	38	33	15	25	17	18
11	do	Silty clay	12	14	36	50	19	30	16	21

[1] In general the use of the term "large" indicates the presence of longitudinal cracking in addition to the usual transverse cracks, and the use of the term "small" the absence of longitudinal cracks.

The shoulders are below the edges of the pavement in places but, in general, the drainage conditions are good

Although the number of subgrade samples is too small to warrant drawing any definite conclusions, attention should be called to certain indications which seem to show that the amount of cracking may be influenced by the character of the underlying soil. For instance, the soil represented by sample No. 3, which showed a very high clay content, 65 per cent, with only 8 per cent of sand and a moisture equivalent of 22 per cent, was taken from beneath the pavement at a point where considerable cracking had taken place. The transverse cracks at this point averaged 15 feet apart with a longitudinal crack, offset slightly from the center on the side from which the sample was taken. Again, sample No. 5, carrying 54 per cent clay, 11 per cent sand, and a moisture equivalent of 21 per cent, was taken from a point under slab No. 4, which had cracked in several places. On the other hand, sample No. 4, which was taken from under the center of slab No. 11, contained only 36 per cent clay as against 45 per cent sand and a moisture equivalent of 15 per cent. At this point no longitudinal crack

(Continued on page 92)

A STABILITY TEST FOR BITUMINOUS MIXTURES[1]

DEVELOPED BY THE DIVISION OF TESTS UNITED STATES BUREAU OF PUBLIC ROADS

Reported by **W. J. EMMONS**, Highway Research Specialist, and **B. A. ANDERTON**, Chemist

THREE years ago the United States Bureau of Public Roads undertook the study of the stability of bituminous paving mixtures. The program adopted embraced both experiments upon especially constructed pavements and investigations in the laboratory using compressed specimens, the objects being to develop a test which would render possible the predetermination of the relative stability of combinations of materials and ultimately to evolve a theory of proportioning.

Late in 1922 a circular track or roadway was paved with 27 asphaltic concrete mixtures of widely varying proportions laid on a smooth concrete foundation. Traffic was applied in the form of a loaded truck, commencing in October. During the ensuing winter all of these mixtures showed no displacement, but with the advent of warm weather shoving began to occur. By the end of July reference screws set in the mixtures showed that movements had occurred in all sections, virtually imperceptible in some and amounting to several feet in others. Although this test was designed primarily to determine relative resistances of mixtures under traffic for the purpose of correlation with the desired laboratory test, it was natural to attempt to draw direct conclusions from their behavior. Such conclusions, however, were confusing and evidently inconsistent.

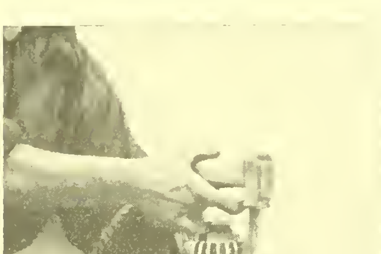


FIG. 1.—Mold and tamping apparatus for use in forming specimens

During the winter of 1923–24 sawed specimens of these mixtures were subjected to compression tests and laboratory investigations were made upon the component sand mortars and coarse aggregates. Results of the latter in connection with the observed action of the track indicated the essential importance of developing the factors influencing the behavior of sand mixtures independently of the presence of the coarse particles. Therefore, in July, 1924, the asphaltic concrete sections were removed and in their place a second series was constructed. This time 33 sections were laid, of which 5 were asphaltic concretes and 28 were sheet asphalts, the customary intermediate binder course under the latter being omitted. Following the installation of measuring devices and thermocouples, traffic was started about the 1st of September and, after being suspended during the winter months, has now been resumed. Several of the mixtures have already been displaced to some extent and it is anticipated that marked differences in their respective resistances to displacement will be shown by all of the sections before the warm months have passed.

LABORATORY TEST DEVELOPED

Meanwhile, considerable has been accomplished in the way of developing a laboratory test by means of which it is hoped to reproduce in terms of laboratory results the demonstrated relative stabilities of the service test mixtures. A test has been developed which differentiates between mixtures of varying composition, and although it is as yet in somewhat tentative form it appears decidedly promising. Results indicate that it is probably applicable to mixtures of both fine and coarse graded types. The test involves the molding of a rectangular slab of the mixture and, by pressure exerted on one side of the confined specimen, extruding a certain portion through openings in the sides and bottom of the mold. Specimens used in this test are 8 inches long, 6 inches high, and 2¼ inches thick. The mixtures are prepared by hand or mechanical mixing and are compressed in a collapsible steel mold, the sides of which are slightly greased. Compression is obtained by means of an electric hammer to which a square-ended tamper is fitted. The hammer being used works on the electromagnetic principle and strikes about 1,500 blows per minute. As the heated composition is fed in small increments, the mold is gradually filled and finally struck off to a level surface by means of a hot iron. Five to eight minutes are required to mold a specimen by this method. Figure 1 shows the apparatus used in the process of forming a specimen.

Although sheet asphalt mixtures can be molded into specimens under gradually applied pressures as imposed by a testing machine, it has been found that adequate compression can not be obtained in the case of asphaltic concretes without fracturing the stone particles. With the trap-rock aggregates used in the tests thus far, this method of tamping has resulted in very little breaking of the aggregate, even when the voids in the specimens made have been low.

The strength or resistance to displacement as measured by the several methods of testing attempted is largely dependent upon the density to which the specimens are compressed. Consequently, at some time previous to testing, the compacted specimen is removed from its mold and its specific gravity determined. From the known specific gravities and the proportions of the constituent materials the percentage of voids in a specimen is calculated.

DISPLACEMENT CAUSED BY SHEARING FORCES

It is apparent that the force which results in the longitudinal displacement of the bituminous surface is a shearing force. When the mixture is compressed under the roller the aggregate particles occupy certain positions with relation to each other, and in combination with the bituminous binder possess to a greater or less degree strength to resist the forces which tend to change this structure. If, under conditions affecting the plasticity of the mixture, forces are applied of sufficient magnitude the aggregate particles move over one another and occupy new positions, giving rise to a different internal structure which may be less strong than the original. With greatly reduced strength waving will ensue, or, under a condition of extreme plasticity, rutting may result. Such a mixture must be regarded as unstable under the traffic, climatic, and other local conditions to which it may have been subjected.

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[1] Presented at meeting of American Society for Testing Materials, held at Atlantic City, June 22–24, 1925.

It is felt, therefore, that the proper test for stability should be essentially one of shear and that the force applied should in some manner cause an internal movement and rearrangement of the particles without leaving them free to dissociate themselves completely during the progress of the test. The expedient of subjecting to pressure a specimen confined in a mold and forcing it through a single restricted orifice seems to fulfill these conditions and appears to be quite satisfactory for the testing of fine-graded mixtures. Considerable experimentation with such a single opening has already been made by other investigators, using varied sand aggregates and fine mixtures. It is felt, however, that this method is not likely to prove successful for use with mixtures containing coarse aggregate since, in specimens of a reasonable size, the forced rearrangement of the original position of the coarse fragments will result merely in the formation of a new and articial internal structure with arching of the particles over the orifice, rendering a determination of the original resistance impossible.

The test here described is intended to be applicable to the entire range of aggregate sizes, and minimizes the objectionable arching action by permitting individual fragments considerable latitude in direction of movement.

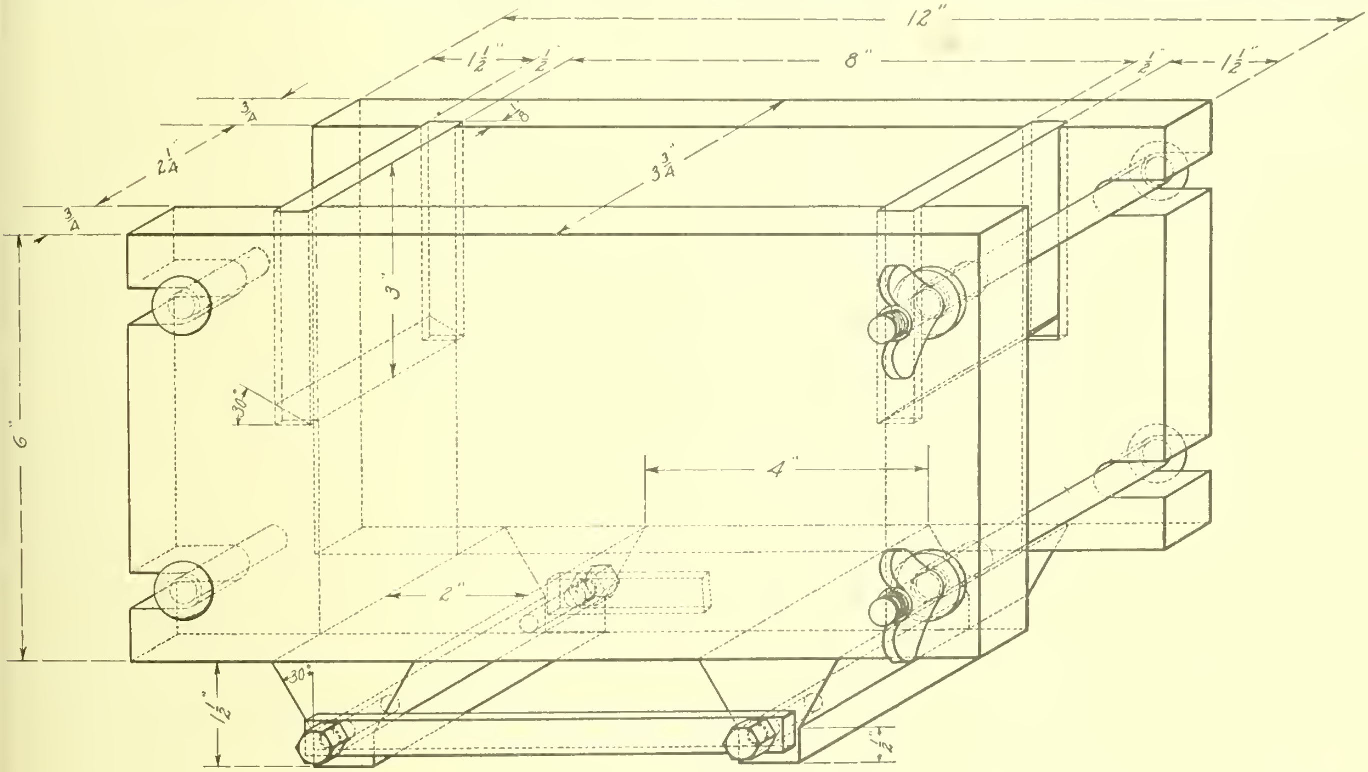

FIG. 2.—Diagram of testing mold and base plates

DESCRIPTION OF THE TEST

The testing form or mold provides three openings for the extrusion of the mixture, the center 4 inches of the bottom and the lower 3 inches at each end. Figure 2 illustrates the construction of the testing mold and its position upon bearing plates during the application of the load. The testing mold is identical with that used in forming the specimen except that full height end plates are necessarily used during that process.

Subsequent to the determination of the specific gravity of the specimen, it is confined in the testing mold and immersed in a water bath maintained at 60° C. A generous time has thus far been allowed to raise the specimen to this temperature, but it is probable that an immersion period of 4 hours is sufficient. The selection of 60° C. as the testing temperature was

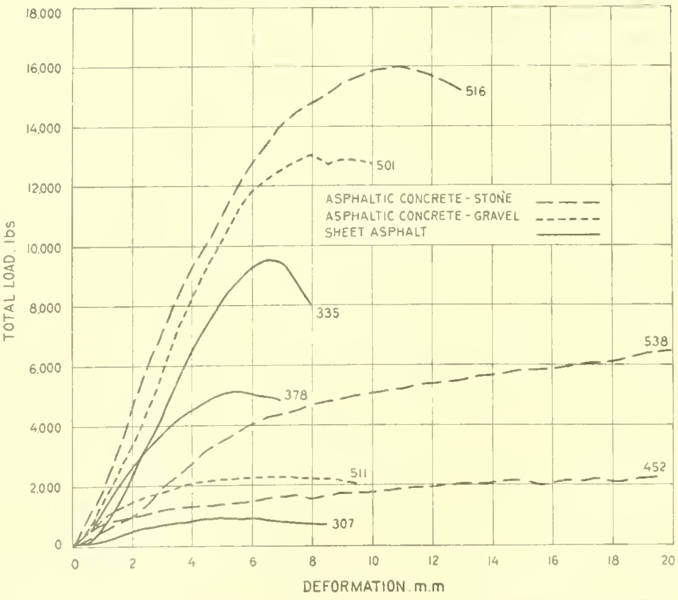

FIG. 3.—Typical stress-deformation curves of asphaltic concrete and sheet asphalt specimens

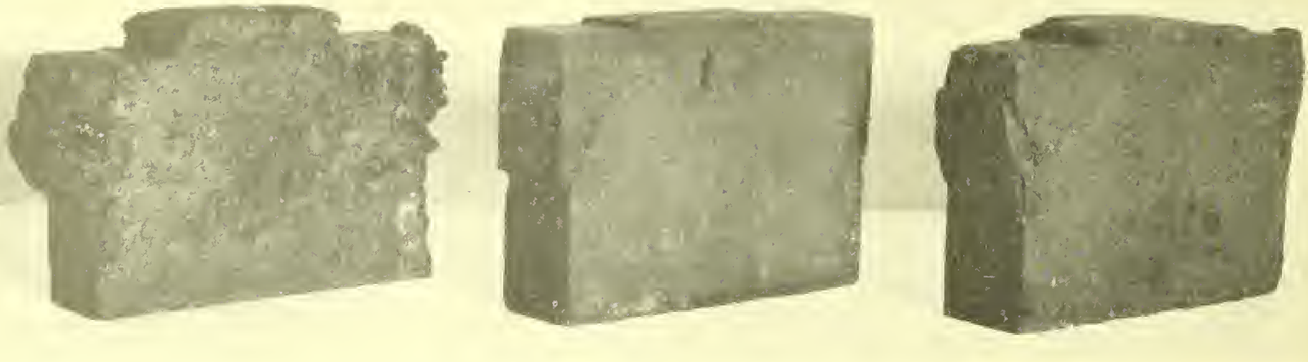

FIG. 4.—Specimens of asphaltic concrete and sheet asphalt after testing

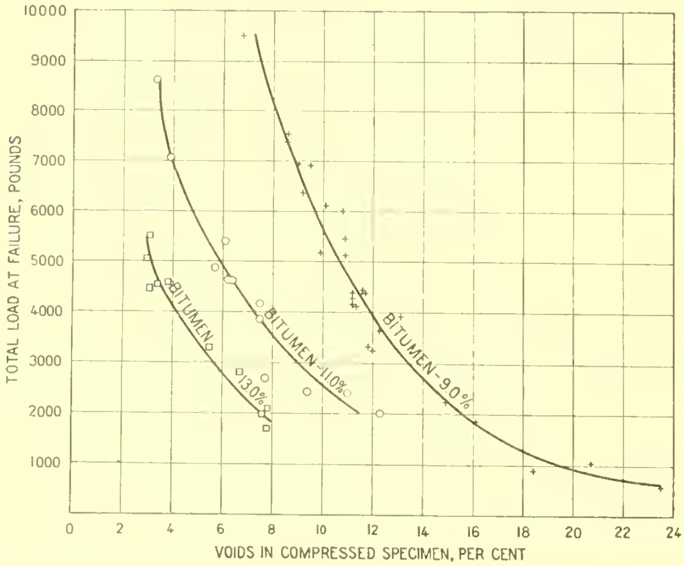

FIG. 5.—Relation between percentage of voids and strength of sheet asphalt containing three percentages of bitumen

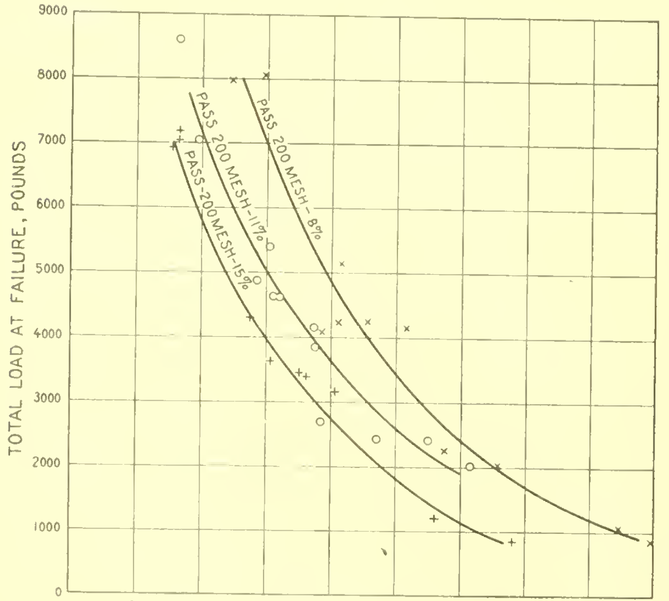

FIG. 6.—Relation between percentage of voids and strength of sheet asphalt containing three percentages of filler

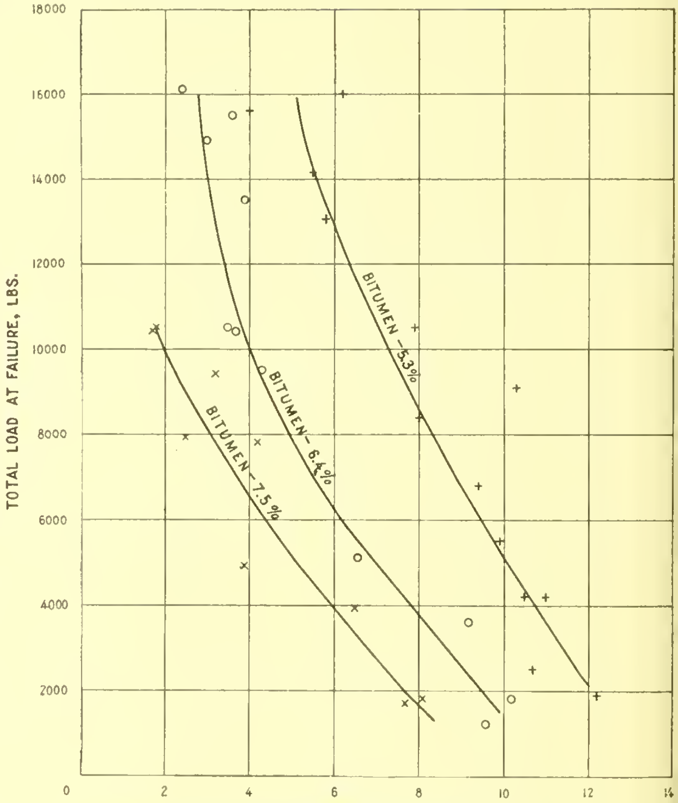

FIG. 7.—Relation between percentage of voids and strength of stone asphaltic concrete with three percentages of bitumen

made as the result of thermocouple measurements of temperatures in the service test pavements. This temperature was reached during extremely hot periods when, of course, the pavement mixtures were also most susceptible to displacement.

When ready for testing the mold containing the specimen is placed upon the properly spaced base plates in a metal tank containing water at 60° C., which is in turn placed upon the weighing table of a 20,000-pound compression machine. The load is applied through a spherical bearing block which rests upon a steel plate having a one-sixteenth inch clearance around the sides of the mold. The machine is run at its slowest testing speed, which with this machine lowers the head at the rate of 0.073 inch per minute. The loads are observed at 0.5-millimeter intervals of deformation.

RELATION BETWEEN STRENGTH AND VOIDS IN BITUMINOUS MIXES

Figure 3 shows typical stress-deformation curves resulting from tests of both sheet asphalt and asphaltic concrete mixtures. In all tests of the former type a very definite maximum load is reached, after which deformation of the specimen progresses under a decreasing load. This statement is also true of the asphaltic concrete mixtures tested thus far, provided that in the process of molding a specimen a good density is obtained. Specimens of this type which are not well compressed frequently do not exhibit definite points of failure, although indications evidenced by periods of continued deformation under constant load are often noted. This situation, however, is not likely to prove disturbing, since it is improbable that much useful information can be obtained from specimens which are not compressed to densities comparable with those attained in construction work. Poorly compressed specimens were, however, purposely made to determine the limits of usefulness of the test being developed. In Figure 3, specimens 452 and 538 were poorly compressed; specimen 516 was molded to a high density. The differences in behavior due to these conditions are clearly shown by the shapes of the curves.

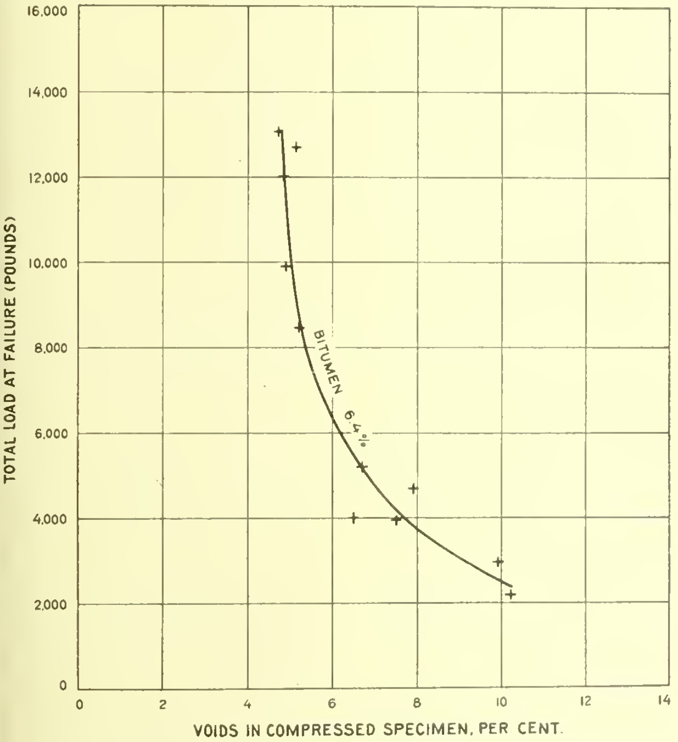

FIG. 8.—Relation between percentage of voids and strength of gravel asphaltic concrete

Figure 4 illustrates the condition of tested specimens after they are removed from the testing molds. They are shown here in an inverted position. The center specimen is composed of a sheet asphalt mixture, while the end ones are of asphaltic concrete. In the vicinity of the mold orifices the internal structure of the tested specimens is entirely disrupted; and when the test is carried as far as was done in the case of the specimen shown at the left, the extruded material crumbles readily. No fracture of coarse aggregate resulting from deformation under test has been noted in the extruded material.

Figures 5, 6, 7, and 8 show the relation between voids in a large number of the specimens and their respective maximum strengths, or, in the case of the lower-density asphaltic concrete specimens, the load at which the form of the stress-deformation curve seems to indicate a reduced stability. The proportions used in these mixtures were varied in such manner as to indicate the sensitivity of the test. A Potomac River sand possessing a grading commonly regarded as suitable for light-traffic sheet asphalt construction was used in all these specimens. Limestone dust served as a filler and a Mexican petroleum asphaltic cement of 55 penetration was used throughout the work. Trap rock graded from 1½ inches to one-fourth inch, but containing a considerable number of fragments whose greatest dimension was 2 inches, was used in the stone asphaltic concrete. Gravel graded between the same limits constituted the coarse aggregate in the series of tests whose results appear in Figure 8.

Figures 5 and 6 indicate differences in the strength of the sheet asphalt mixtures resulting from changes in the bitumen and filler contents, respectively. Figure 7 shows the effect of variations in bitumen content upon a certain asphaltic concrete. In these latter mixtures the bitumen contents were computed by assuming that the stone required 1.25 per cent of bitumen and that the sand and dust carried 9, 11, and 13 per cent of bitumen in the three series indicated as containing 5.3, 6.4, and 7.5 per cent of bitumen, respectively, thus conforming to the percentages used in the three sheet asphalts shown in the curves of Figure 5. The total aggregate mixture of the asphaltic concrete specimens is composed of equal parts retained upon and passing a 10-mesh sieve. The specimens of the single series of gravel asphaltic concrete are of the same proportions as the stone asphaltic concretes of Figure 7, which contain 6.4 per cent bitumen.

These results should not be taken as definite measures of the stability of mixtures so proportioned. Data thus far obtained are much too meager to be so used, and the correlation between the behavior of the service-test sections previously referred to and the results of the strength tests remains to be accomplished before definite interpretation of the results can be given and the true value of the test confidently asserted.

TABLET COMMEMORATES VISIT OF LATIN AMERICANS

A bronze tablet commemorating the visit to the United States of delegates to the Pan American Highway Commission was unveiled at the Pan American Union in Washington on June 16.

In presenting the tablet Ambassador Mathieu of Chile said: "It is intended to express the deep appreciation and gratitude of the delegates from the republics of Latin America for the many courtesies and kindnesses shown them in the United States, and especially for the inspiration and stimulus which they derived from their visit to this great country. The inspection which they made not only convinced them of the marvelous progress of road building in the United States but also kindled their enthusiasm to foster similar movements in their respective countries.

EFFECT OF GRADING ON SAND STRENGTH RATIOS

BY THE DIVISION OF TESTS, UNITED STATES BUREAU OF PUBLIC ROADS

Reported by **C. E. PROUDLEY**, Assistant Engineer of Tests

THE quality of concrete sand has been determined by means of the strength ratio test for a number of years, in some cases by means of compression tests, but more frequently by means of tension tests. It has been noticed, however, that the two methods do not agree for determining the quality of the same sample of sand. The discrepancy has been so great in some instances as to make it possible for a sand to meet requirements for quality when tested by one method and fail if the other method is used.

The continued occurrence of such discrepancies has led to a study of the data available in the physical laboratory of the Bureau of Public Roads for the purpose of finding the factors influencing the relationship between the two methods. The first study which is described in this article embraces the relation between quality and grading as it exists for a large variety of sands which have been used or proposed for use in concrete. There is no thought, as a result of this study, of establishing either method as most suitable for indicating quality. Each method has its advantages and one test is probably as suitable as the other.

The most noteworthy observations to be drawn from this analysis are that coarse grading of sand results in an increase of the compressive strength ratio as compared to the tensile strength ratio and in general the tensile strength ratio test is more favorable to sand than the compressive strength ratio.

The data for the comparisons were obtained from routine tests made in the laboratory during the past three years. In all there were about 180 materials represented, most of which were considered satisfactory for concrete work, and some of which were doubtful. All were natural sands and covered a complete range of common types.

Reference to American Society for Testing Materials Standard C9–21, Section XIV, will give a description of the method of making, storing, and testing the tension specimens. Tentative standard method C9–16T, of the American Society for Testing Materials, describes the making of compression tests, which is essentially the same as performed in this laboratory. It should be noted here that the consistency of the natural sand mortar was made as nearly the same as that of the Ottawa sand mortar as the operator could judge by close observation and feeling. Inasmuch as five operators took part in this series of tests, it is probable that the influence of the personal factor is comparatively small.

For the purposes of this investigation, it was thought best to average the 7 and 28 day strength ratios and use these figures as the quality factor of the sand. The ratio of the tensile strength factor to the compressive strength factor was next computed, in order that those materials having the same relationship of quality factors could be grouped. This means that the materials having higher tensile strength ratios than compressive strength ratios were put in one group, those having approximately the same strength ratios in another group, and so on. The mechanical analyses of each of these groups were averaged, thereby obtaining the figures shown in Table 1.

These relationships are shown graphically in Figure 1 by plotting the grading number for each sample, which is the summation of the percentages retained on the 10, 20, 30, 50 and 100-mesh sieves against the ratio of the tensile strength ratio to the compressive strength ratio. The average curve drawn on this plot was obtained by averaging the ratios in vertical columns and drawing a smooth curve through these points. This gives the average relationship between tensile and compressive strength ratios for any given grading number. The wide dispersion of the points makes this curve of little practical use other than to indicate the probable ratio of tensile strength ratio to compressive strength ratio. It is interesting to note that the average variation of this relationship from the curve is 14 per cent.

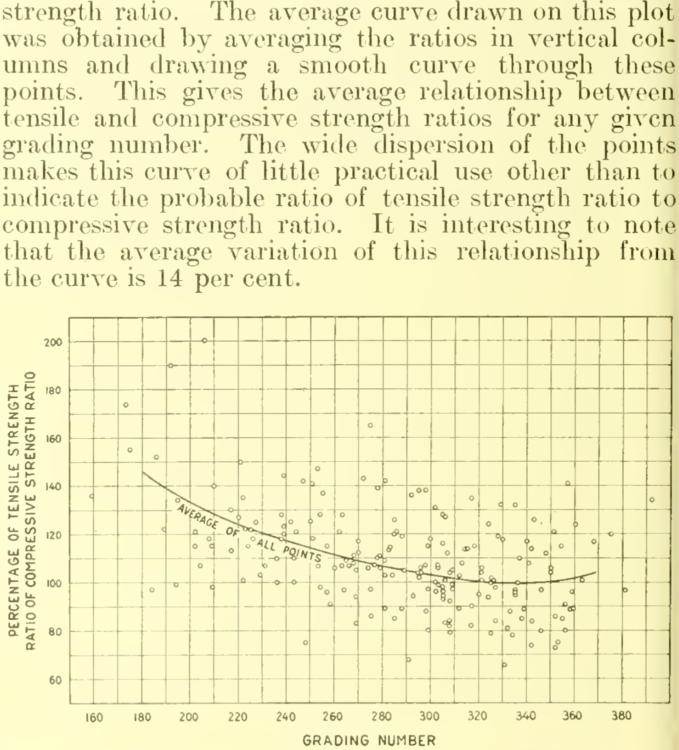

FIG. 1.—The average curve shows that an increase in the coarse grading of the sand results in an increase of the compressive strength ratio as compared to the tensile strength ratio.

Undoubtedly much of this variation is due to improper manipulation of the compression tests for none of the operators had had extensive experience in molding the 2 by 4 inch cylinders prior to these tests. Other factors likely to cause the dispersion of points noted are the shape of the sand grains, organic matter, and structural weakness of the grains. None of these factors can be accurately measured at present and consequently no attempt has been made to consider their influence. The use of sand from any one source graded artificially and used in a series of tests of this nature would eliminate nearly all of the incidental variables encountered in this study.

TABLE 1.—*Effect of grading of sand upon the relation between the tensile and compressive strength ratios*

Ratio of tensile strength ratio to compressive strength ratio	Number of samples averaged	Mecbanical analysis						
		Retained on sieve No.						Loss by elutriation
		10	20	30	50	100	200	
Per cent		*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*
75– 85	20	22	45	67	89	96	98	2.1
86– 95	20	20	42	67	90	96	98	2.0
96–105	37	15	38	65	88	96	98	2.2
106–115	42	13	32	57	86	96	98	2.2
116–125	26	12	30	55	84	95	98	2.0
126–135	14	17	34	58	86	96	98	1.9
136–145	13	10	27	53	81	94	97	2.7

Not many laboratories are so situated that only a single source of sand need be considered. There are usually from 4 to 25 different materials available, each of which may show changes in character as they are placed on the market. The variations shown here, therefore, would prevail to a greater or lesser degree in nearly all localities. In other words, the conditions of general relationship shown on the chart are applicable in the majority of testing laboratories which operate over an extended area.

Inspection of the table shows that in those instances where the tensile strength ratio is lower than the compressive the sands average coarser in grading than where the tensile strength ratio is the higher. This is shown also on the chart. The low grading numbers indicate fine grading and at this end of the curve the ratio numbers are high—that is, the tensile strength ratio greatly exceeds the compressive strength ratio in amount.

Also it is to be noted that the average curve drops to about the 100 per cent line, where tensile and compressive strength ratios are equal, and then curves upward. This is equivalent to saying that the tensile strength probably will not fall below the compressive strength ratio. The reversal of direction of the curve is probably due to several coarse sands with higher percentages of voids than the others. It seems that in molding briquettes of exceptionally coarse sands more attention is given to the compaction of the mortar than in the molding of cylinders. The result is a denser and consequently stronger briquette and therefore a more favorable tensile strength ratio.

NEW MOTION PICTURE RELEASES OF THE BUREAU OF PUBLIC ROADS

Two new motion pictures of road building have been prepared for the Bureau of Public Roads by the Office of Motion Pictures, United States Department of Agriculture, and these are now ready for distribution. Both show the economic and scenic values of the roads in the National forests in the Pacific Northwest and the State of California.

"The Road Goes Through" is a picture of National forest road construction, although the settings are of such a scenic nature that the subject matter would be of interest to any audience. Some of the views are: Crescent Lake in Washington—a trout fisherman's paradise; a bridge over a glacial torrent on Mount Rainier; highway engineers surveying the Coast Highway along the cliffs in Oregon; road crews at work on the Mount Hood Loop road in Oregon; teams excavating in the big tree section of California; a surveyor felling a big timber and a donkey engine clearing right of way in the "blow-down" area of the Olympic Peninsula, Wash. Gyratory rock crushers, motor trucks, road planers, blasting crews, wheel scrapers, road camps, steam shovels, and other equipment may be seen in actual use and always in a rugged and spectacular background. The picture throbs with life and will be of interest to any group regardless of age or occupation.

"Roads from Surf to Summit" has a background which is entirely scenic and gives some conception to the prospective automobile tourist of the economic resources and the hidden scenic treasures in this country. Mount Rainier, Wash., is seen with its snow capped head touching the clouds at an elevation of 14,408 feet above sea level. There are views of the Coast Highway along the rugged, rocky, and precipitous coast of Oregon, and of Mount Hood, probably the most accessible major peak in this country. The 173-mile loop road around Mount Hood has been made possible by the completion of a 37½-mile section of forest road on the southerly slope of this mountain. The road reaches an altitude of 4,735 feet above sea level and the traveler returns to Portland over the paved Columbia River Highway through the river's gorge in the Cascade Mountains.

A description of the bureau motion pictures now ready for release follows:

NEW PICTURES.

Roads from Surf to Summit.—1 reel (872 feet).

Economic resources and scenic wonders of the national forests of the Pacific Coast States now revealed to the motorists by means of modern highways constructed under the direction of the Bureau of Public Roads. Includes Lake Crescent in the Olympic National Forest, the big timber, Mount Hood, Mount Rainier, Mount Baker, Mount Shuksan, and many other natural resources and wonders of the West.

The Road Goes Through.—1 reel (911 feet).

How the western road builder overcomes barriers to transportation and builds the modern roads of our national forest and Federal-aid highway systems.

FOREST ROADS.

Roads to Wonderland.—1 reel (851 feet).

Scenic spots reached by roads that are being built by the Federal, State, and county governments; Mount Hood, in the Mount Hood National Forest; Crater Lake, in Crater Lake National Park; and Yosemite National Park. Of general interest.

Highroads and Skyroads.—1 reel (907 feet).

Building Government roads through the national forests; obstacles overcome and scenic beauties reached. Of general interest.

Around the West by Forest Roads.—1 reel (985 feet).

Examples of forest roads built by the Bureau of Public Roads in Colorado, Oregon, California, and Arizona. Of general interest.

Building Forest Roads.—1 reel (959 feet).

Men and machinery at work in the national forests, pushing good highways through the great mountains and woodlands. Of general interest.

ROAD BUILDING.

Modern Concrete Road Construction.—1 reel (995 feet).

Approved methods of highway building by the use of concrete; some of the modern machinery and practices used in this work; a contrast is drawn between old-fashioned mud roads and modern highways; the film ends with a race between a railway train and a motor truck on a concrete road. Of general technical interest.

Mixed Asphalt Pavements.—1 reel (892 feet).

Construction of asphaltic concrete and sheet asphalt roads as approved by the Bureau of Public Roads in administering the Federal-aid road act, from asphalt plant to finished road. Of general technical interest.

Building Bituminous Roads.—1 reel (772 feet).

How Uncle Sam, as the world's greatest road builder, constructs surface-treated and penetration macadam roads. The introductory scenes show sources of bituminous materials and laboratory tests to determine quality. Of general interest.

Brick—From Clay to Pavement.—1 reel (960 feet).

The progress of clay through the plant until it is laid on the road as vitrified paving brick; some finished brick roads. Of general technical interest.

What About Macadam?—1 reel (836 feet).

Approved methods of building macadam highways on Federal-aid projects. The introductory scenes explain why macadam roads may not be satisfactory for automobile traffic, and how to make them so. Of general technical interest.

Granite Block Paving.—1 reel (706 feet).

From the quarry to the finished pavement in large cities. Of general technical interest.

Tests for Better Roads.—1 reel (926 feet).

Tests conducted by the Bureau of Public Roads to determine the durability of road building materials; some of the unusual machinery used in making the tests. Of general technical interest.

The general public may obtain the films by written application to the Office of Motion Pictures, Extension Service, United States Department of Agriculture, Washington, D. C. Films are furnished free except for the transportation charges both ways. In all cases it is necessary that a reliable person assume responsibility for such charges as well as for the safe keeping, proper use, and prompt return of the films. Applications should be made as far in advance as possible and should specify preferably several alternative choices of subjects and periods of time. Schedules of proposed showings or other definite information indicating the prospective use should accompany the application. About one-half the available prints are on slow burning stock and this is being used for all new films. All films are of standard width.

(Continued from page 85)

had developed, and the transverse cracks were from 10 to 20 feet apart. All of these samples were taken from the subgrade in Muskingum County.

Turning now to Licking County, we find that sample No. 6 was taken from a section where a longitudinal crack had formed and where two transverse cracks extending partly across the pavement had developed only a few feet apart. This sample, as will be noted by reference to the table, shows a clay content of 47 per cent, a sand content of 18 per cent, and a moisture equivalent of 22 per cent. It will also be noted that the dye adsorption of 38 on this soil was the highest of any of the samples tested. The other four samples taken from Licking County were obtained at locations where little cracking had developed. Two of these samples, Nos. 7 and 10, show relatively low clay content and moisture equivalent, with corresponding high sand content. The other two, however, show high clay content and low sand content. In the last-mentioned cases, however, samples Nos. 8 and 11, it will be seen that both samples contained a considerable amount of coarse material (gravel), which may possibly have increased the stability of the soil, and reduced the amount of cracking. It will be noted also that in the case of all four of the samples from Licking County taken from under the concrete which had not cracked extensively, the percentage of shrinkage of the soil as determined by laboratory test was lower than in the case of sample No. 6. In general, it may be said that the results of the soil tests appear to bear a distinct relation to the amount of cracking in the concrete.

PRACTICALLY NO SCALING OF SURFACE

The road is practically free from surface scaling due, in the writer's opinion, first, to the fact that the sand used in the concrete contained very little fine material; second, to the fact the concrete was mixed dry; and third, that there was no tendency to overfinish the surface. Experience has demonstrated that scaling is very apt to occur on pavements which have been overfinished in an effort to secure an exceptionally smooth surface and that this condition is exaggerated when the concrete is wet and when the fine aggregate contains a relatively large amount of silt. Some surface wear, however, has taken place on the pavement. This wear is uniform across the road and has taken place to about the same extent on both the uncrushed gravel and crushed-limestone concrete. The total amount of wear which has taken place in 10 years of service, as near as can be estimated, is about one-eighth of an inch. The wear, however, has not been accompanied by an undue amount of spalling or disintegration of the concrete. This applies to the small slabs which have been formed by the excessive cracking of the gravel concrete and as well as to the edges of exposed joints or cracks. In only a very few cases has it been necessary to replace slabs due to the breaking up of the concrete and this again is a tribute to the quality of the concrete. From present indications, the pavement should be good for many years' service, assuming that proper maintenance is given the joints and cracks.

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ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage in the United States, 1914. A Summary.
*393. Economic Surveys of County Highway Improvement. 35c.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583. Report on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916. 10c.
*660. Highway Cost Keeping. 10c.
*670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691. Typical Specifications for Bituminous Road Materials. 10c.
*704. Typical Specifications for Nonbituminous Road Materials. 5c.
*724. Drainage Methods and Foundations for County Roads. 20c.
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
No. 1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1259. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1279. Rural Highway Mileage, Incomes and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULAR

No. 94. TNT as a Blasting Explosive.

FARMERS' BULLETINS

No. *338. Macadam Roads. 5c.
*505. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *727. Design of Public Roads. 5c.
*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

MAY 31, 1925

STATES	FISCAL YEARS 1917-1924			FISCAL YEAR 1925									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1924			PROJECTS COMPLETED SINCE JUNE 30, 1924			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 4,598,721.63	$ 2,186,247.54	464.1	$ 1,371,376.08	$ 676,950.32	147.7	$ 15,450,912.60	$ 7,494,856.47	804.8	$	$		$ 2,436,778.67	Alabama
Arizona	8,338,365.41	4,287,683.88	527.8	978,083.89	567,297.92	78.1	2,278,241.44	1,382,630.30	149.8	449,594.93	293,904.22	46.6	2,020,355.68	Arizona
Arkansas	11,094,751.31	4,424,345.63	944.4	2,142,283.34	919,258.39	98.9	7,063,606.29	3,055,965.83	387.9	313,994.94	124,097.83	16.7	1,802,896.32	Arkansas
California	12,999,075.03	5,647,148.17	533.7	8,878,662.25	4,795,835.51	341.6	9,499,195.59	4,573,905.00	243.9	66,685.30	39,173.75	0.2	4,509,879.57	California
Colorado	8,108,070.31	4,029,898.97	502.6	2,485,167.72	1,327,686.59	108.3	4,700,294.22	2,494,541.04	195.9	732,123.20	279,865.34	32.2	2,801,126.06	Colorado
Connecticut	3,062,872.02	1,269,558.60	73.6	1,128,828.24	438,051.93	21.9	2,358,530.17	720,572.55	37.1				1,427,812.92	Connecticut
Delaware	3,066,832.22	1,007,714.83	72.5	1,224,727.59	488,475.82	34.6	489,503.74	213,581.10	12.3	639,364.40	244,183.75	14.1	151,199.50	Delaware
Florida	961,134.07	461,470.92	48.8	1,998,139.65	944,017.05	47.5	8,787,223.95	4,279,162.51	251,8	116,547.09	91,547.09	0.1	1,403,567.43	Florida
Georgia	17,167,373.32	7,956,805.20	1214.2	2,988,629.05	1,450,561.26	264.1	11,050,944.65	5,445,883.28	736.1	439,358.35	186,405.61	16.0	1,394,330.65	Georgia
Idaho	8,161,697.92	4,092,395.52	506.8	1,212,978.88	692,936.74	93.3	2,150,202.17	1,363,194.37	135.7	326,332.23	217,941.33	34.1	1,248,171.04	Idaho
Illinois	26,964,706.06	12,279,546.33	804.7	12,546,850.31	6,116,185.50	413.1	7,776,339.53	3,767,193.20	264.3	961,985.29	475,577.53	34.3	3,989,468.44	Illinois
Indiana	7,577,444.16	3,655,540.97	225.7	5,597,232.07	2,704,844.77	185.4	11,706,762.88	5,704,563.55	378.6	421,857.03	210,928.56	14.3	3,975,207.15	Indiana
Iowa	23,195,778.19	9,237,031.86	1682.9	4,020,893.35	1,842,654.30	290.3	5,588,867.96	2,534,660.17	391.9	2,779,997.22	1,188,376.05	137.6	2,603,810.62	Iowa
Kansas	17,084,136.48	6,043,176.80	502.7	9,140,903.95	3,625,812.15	308.1	12,757,239.68	5,443,788.88	620.0	728,033.69	201,106.21	13.7	2,059,764.96	Kansas
Kentucky	10,822,980.31	4,613,947.28	429.4	3,597,161.74	1,436,148.07	133.7	8,028,463.36	3,589,978.95	288.9	474,921.70	237,460.83	36.7	1,905,810.87	Kentucky
Louisiana	8,488,463.18	3,636,143.36	661.2	3,450,961.79	1,643,727.50	266.5	3,363,645.07	1,620,766.20	162.7	145,263.04	72,631.51	0.1	1,289,435.43	Louisiana
Maine	6,911,058.78	3,299,935.38	230.7	1,263,222.53	607,934.95	50.7	749,878.17	350,363.39	24.2	114,562.51	57,281.25	5.3	1,459,597.03	Maine
Maryland	6,760,044.42	3,213,321.78	243.2	1,307,190.18	606,526.37	49.2	1,909,803.12	807,640.52	56.7	1,393,774.91	528,956.13	86.3	128,288.20	Maryland
Massachusetts	10,191,202.02	4,105,727.22	232.8	3,640,547.27	1,259,604.71	62.9	4,871,161.33	1,355,248.12	77.3	568,395.79	153,810.00	8.5	2,135,507.95	Massachusetts
Michigan	13,434,135.07	6,060,612.23	494.5	2,472,673.22	1,110,505.92	106.0	14,963,098.54	7,078,061.87	481.8	352,277.44	171,034.19	5.7	3,684,784.79	Michigan
Minnesota	24,037,561.24	9,885,843.07	2292.0	6,378,124.65	2,852,798.97	429.2	6,490,887.56	3,127,400.00	670.8	1,587,176.37	410,000.00	83.3	1,166,527.96	Minnesota
Mississippi	7,888,193.89	3,828,941.39	655.0	2,191,938.63	1,056,444.54	148.2	8,738,641.87	4,363,065.28	482.7	220,062.43	110,031.20	7.5	1,464,750.59	Mississippi
Missouri	11,352,027.70	5,245,899.18	803.5	5,640,412.83	2,786,985.58	301.4	26,137,702.67	10,506,055.15	856.4	1,334,791.25	414,019.00	38.6	1,404,957.09	Missouri
Montana	8,867,279.16	4,384,335.12	791.4	1,289,321.25	933,188.03	130.3	1,882,291.46	1,379,095.35	165.1	1,057,077.93	600,720.40	119.9	5,217,551.10	Montana
Nebraska	7,876,337.16	3,714,691.59	1440.4	1,348,825.26	635,545.10	114.1	8,125,476.71	4,023,341.60	850.2	784,267.01	389,327.59	46.1	4,270,009.12	Nebraska
Nevada	3,460,245.52	1,853,624.98	225.6	1,446,557.71	1,228,225.07	131.4	4,836,660.07	4,081,664.43	405.8	374,257.39	328,156.54	61.4	346,725.98	Nevada
New Hampshire	3,076,750.19	1,487,867.58	171.3	1,088,937.67	498,359.29	36.8	812,945.64	391,481.51	26.0	102,182.11	50,992.79	4.1	371,887.83	New Hampshire
New Jersey	7,623,795.12	2,661,531.49	148.7	4,113,182.90	1,120,958.50	67.8	8,335,555.49	2,455,827.11	52.6	509,417.17	110,955.00	7.4	1,175,056.90	New Jersey
New Mexico	5,306,286.46	2,758,849.68	714.3	3,256,393.66	2,059,699.71	351.0	4,524,906.13	2,988,419.80	410.6	250,983.39	154,354.77	16.7	1,813,174.04	New Mexico
New York	18,862,742.49	8,257,844.44	572.7	7,568,694.12	3,102,822.01	201.6	30,368,096.49	10,680,589.16	644.3	5,711,500.00	1,498,931.25	99.3	6,825,057.14	New York
North Carolina	12,567,732.97	5,676,757.66	884.7	6,374,361.39	2,314,976.07	177.6	8,939,651.62	3,730,868.46	251.2	1,976,524.06	887,886.59	50.6	1,382,930.22	North Carolina
North Dakota	9,088,973.11	4,418,506.42	1587.9	1,707,265.90	833,931.18	325.6	2,766,616.44	1,394,255.38	391.8	597,953.78	320,630.51	65.1	2,577,032.51	North Dakota
Ohio	33,122,751.43	11,879,917.99	962.5	8,105,457.77	3,228,875.94	218.8	9,551,814.30	3,416,129.95	284.8	2,283,493.20	1,043,230.22	73.6	3,361,597.90	Ohio
Oklahoma	12,986,865.26	5,888,852.03	497.3	6,967,406.16	3,388,195.41	310.5	5,895,010.08	2,742,551.82	259.8	2,910,706.83	1,320,204.58	143.7	952,004.16	Oklahoma
Oregon	12,082,873.17	5,819,093.79	655.6	2,162,545.40	1,274,243.08	140.2	2,773,422.07	1,602,718.08	127.4	730,412.11	472,088.22	42.6	517,683.83	Oregon
Pennsylvania	36,825,248.98	14,114,6[illegible]4.7[illegible]	729.7	5,616,297.45	1,952,029.32	110.3	27,232,590.52	7,277,609.43	469.8	10,380,760.11	2,837,639.27	200.3	1,779,766.19	Pennsylvania
Rhode Island	1,774,397.25	779,227.96	46.0	854,098.95	340,460.13	18.8	1,798,608.99	518,589.59	26.3	87,634.25	20,100.00	1.3	640,288.32	Rhode Island
South Carolina	9,016,476.73	4,124,045.22	924.4	1,822,999.07	846,189.44	280.6	6,830,945.32	2,615,520.84	402.3	266,706.71	82,996.62	15.9	1,071,342.88	South Carolina
South Dakota	8,674,597.86	4,244,636.27	989.8	3,367,308.41	1,720,478.53	447.8	6,538,945.37	3,211,410.37	904.9	923,685.61	432,198.51	101.0	324,976.32	South Dakota
Tennessee	6,805,683.35	3,313,936.07	259.6	8,983,457.63	3,418,143.70	238.3	11,932,658.52	5,345,527.55	422.3				1,570,014.68	Tennessee
Texas	42,341,998.56	16,130,624.91	3122.8	10,995,266.74	4,525,401.28	710.5	23,580,295.22	9,821,992.12	1468.8	3,643,542.26	1,449,369.15	208.3	4,152,540.54	Texas
Utah	3,304,423.75	1,895,805.92	219.0	2,714,473.61	1,750,151.47	180.6	2,948,312.94	1,998,339.44	214.3	505,962.57	375,279.38	51.4	943,363.79	Utah
Vermont	1,922,114.16	942,769.12	74.4	924,591.66	435,734.27	28.6	1,347,956.36	642,288.36	30.6				878,812.26	Vermont
Virginia	10,035,301.48	4,801,782.43	562.5	3,064,418.53	1,470,215.77	113.7	9,936,608.82	4,498,851.97	343.0	1,163,378.53	522,593.85	40.7	749,221.98	Virginia
Washington	11,384,615.67	5,290,895.45	457.0	1,842,601.60	781,098.36	66.2	2,769,626.50	1,329,400.00	120.5	652,246.53	274,000.00	16.6	1,330,271.19	Washington
West Virginia	5,489,747.95	2,365,041.53	255.6	1,853,452.91	865,251.80	71.1	4,864,235.15	2,024,363.78	137.4	2,370,340.51	903,966.53	61.7	392,804.36	West Virginia
Wisconsin	18,753,903.15	7,441,033.57	1325.3	3,001,359.02	1,452,667.68	121.0	2,764,923.54	1,320,055.36	135.5	1,496,912.73	767,955.00	78.3	4,570,047.39	Wisconsin
Wyoming	6,127,625.61	3,078,098.70	687.6	2,629,027.49	1,626,626.97	290.5	3,180,085.69	1,987,157.25	152.7	679,528.24	436,798.00	113.1	493,617.08	Wyoming
Hawaii							342,277.22	97,440.00	6.5				633,810.00	Hawaii
TOTALS	549,655,391.27	237,852,399.82	32452.9	176,655,291.47	81,754,712.97	8864.4	371,791,663.21	162,848,566.44	16416.0	53,692,572.14	20,988,704.15	2251.0	94,805,616.62	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 91,433,196.89 Federal aid $ 40,763.046.11 Miles 3,833.4

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A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 6, NO. 5 ▽ JULY, 1925

Type of Milk Tank Mounted on Semi-Trailer Similar to That Used in Chicago Dairy District

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FIELD METHODS USED IN SUBGRADE SURVEYS

BY THE U. S. BUREAU OF PUBLIC ROADS ON THE PACIFIC HIGHWAY, WASHINGTON

Reported by A. C. Rose, Associate Highway Engineer

THERE is scarcely a road of any considerable length anywhere but has for its subgrade at various points a variety of soils differing more or less widely in the physical properties which affect the condition and supporting value of the subgrade. Minor differences may be found from foot to foot of the length of any highway; but generally it will be found that a soil of the same general type prevails for a considerable distance within which the general characteristics of the subgrade are fairly uniform.

The characteristics which it is most important to determine in considering the suitability of any soil to serve as a highway subgrade seem to be the degree to which it will absorb and hold water and the amount of swelling or shrinking which occurs when water is added to or taken from it. These characteristics are ascertained in the laboratory by tests applied to samples of the soil to determine its moisture equivalent percentage and volumetric shrinkage.[1] They may be approximately determined in the field by means of simplified tests developed by the writer in connection with a study of subgrade soils in the Pacific Northwest and described in a previous issue of this magazine.[2] Attention is called to the fact, however, that the shrinkage factor as determined by the laboratory and the field tests are not comparable, since in the former case the moisture content of the soil is the capillary saturation, while in the latter case it is the moisture equivalent percentage.

These tests have been found to be very helpful in establishing the characteristics of doubtful soils; but they are not the only means employed, nor are they even the most important of the methods by which the characteristics of a highway subgrade are determined. The first step in a subgrade survey is to establish the location of the various soil types. Until this is done the selection of representative test samples is impossible, and the results of tests made can be applied only to the particular location from which the samples are taken. But when the soil types are identified and the points at which the soil changes from one type to another determined, it then is possible to select samples representative of each type, and the test results may be considered as applying to all parts of the road in which the sampled types of soil are found.

UNITED STATES BUREAU OF SOILS SURVEYS VALUABLE TO THE HIGHWAY ENGINEER

The greatest aids the highway engineer has for making these subgrade surveys are the publications and maps of special areas which are prepared by the United States Bureau of Soils. These are printed in the form of bulletins and give descriptions of the origin, color, physical characteristics, drainage conditions, etc., of the soils and also show a typical mechanical analysis for each class. The series names used in designating the soils, such as Aiken, Olympic, etc., indicate the origin of the parent material from which the soils are derived, while the type names, such as sand, silt-loam, silty clay loam, clay, etc., are based principally upon the relative percentages of the various constituents of the soil as shown by the mechanical analysis.

For certain areas detailed soil bulletins have been prepared; in others only reconnaissance surveys have been made; and in still other sections no surveys have been carried on. The more definite the information, the easier it is to carry on a subgrade survey. When a detailed and recent bulletin of an area is available it is a comparatively easy matter to walk over a road project and establish the location of the soil types from the colored areas on the soil map, the location of the road and other physical features such as streams and lakes on the map, and the accompanying description in the bulletin.

When reconnaissance surveys only may be had the difficulty is considerably increased. The object of these surveys was to cover a large territory at a small expense. Therefore the detailed soil types on heavily timbered sections may not be shown, and it may also be found that small areas of some types which are large enough to affect highway subgrade may be omitted. In other cases the accurate parts of the maps may be confined largely to the sections productive from an agricultural standpoint. Areas covered by such reconnaissance surveys require detailed investigation in order to identify small areas of soil types which are not shown separately on the map.

The worst condition, however, is presented by the unmapped areas. In these it is necessary to study adjacent areas which have been mapped and extend the nomenclature to the unmapped area under consideration or better yet to secure from one of the agricultural colleges the assistance of an expert in soil mapping who will establish the soil series and type names found in the unmapped section.

If only a short road project is to be investigated the soil maps are useful for determining the several types but the series names need not be established unless it is expected to extend the survey over a large area or over an entire State highway system.

INFORMATION GIVEN BY BUREAU OF SOILS BULLETINS

The Bureau of Soils survey bulletins show the topography and drainage, the general climatic conditions, the geological history of the area and give a classification of soils as to series and types. The description of the soil series shows the color of the soil and subsoil, the nature and origin of the parent material, the topography and drainage where found, and the general characteristics as well as the natural vegetation usually found on the soil. The mechanical analyses determine the classification of the series into types based upon the percentages of the sand, silt, and clay constituents. All of this information presented in the bulletins is useful in identifying the soils in the field.

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[1] For a description of the moisture equivalent and volumetric shrinkage tests as made in the laboratory see PUBLIC ROADS, vol. 6, No. 2, April, 1925.

[2] Practical Field Tests for Subgrade Soils. PUBLIC ROADS, vol. 5, No. 6, August, 1924.

THE SENSE OF FEELING A VALUABLE AID IN DETECTING SOIL TYPES

In making subgrade surveys it is a decided advantage to the investigator to be able to check his estimates by rubbing samples of the soil between the fingers. It is remarkable how accurately a specimen may be classified after the operator has had some experience in this work; and, because of the importance of this method for reducing the cost and time of the work, it may be well to discuss it here at some length.

As a rule sands feel gritty; silts, velvety, smooth, and floury; and clays, which are smooth and plastic, offer considerable resistance to pressure, roll into balls when moist and adhere to the fingers. By the degrees in which they combine these salient characteristics of the primary constituents, soils other than pure sands, silts, and clays may be distinguished as sandy loams, loams, silt-loams, sandy clays, clay-loams, silty clay-loams, and silty clays. The writer has never become sufficiently expert in this work to be confident of the estimates made of some of these gradations where the degree of variation is slight. Proficiency is acquired only by instruction from those who have learned the art, but the following notes by Prof. Charles F. Shaw [3] will be helpful.

Sand.—Sand is loose and granular. The individual grains can readily be seen or felt. Squeezed in the hand when dry, it falls apart when the pressure is released. Squeezed when moist, it will form a cast, but will crumble when touched. Sands are classified as coarse, medium, fine, or very fine sands, depending on the size of the grains that compose them.

Sandy loam.—A sandy loam is a soil containing much sand but which has enough silt and clay to make it somewhat coherent. It has a gritty feel, and the sand grains can readily be seen. Squeezed when dry, it forms a cast which can easily be broken, but if squeezed when moist a cast can be formed that will bear careful handling without breaking. Sandy loams are classed as coarse, medium, fine, or very fine sandy loam, depending on the size of the grains of which they are formed.

Loam.—A loam is a soil having an even mixture of the different grades of sand and of silt and clay. It is mellow with a somewhat gritty feel, yet fairly smooth and rather plastic. Squeezed when dry, it forms a cast that will bear careful handling, while the moist cast can be handled quite freely without breaking.

Silt-loam.—A silt-loam is a soil having little sand and only a small amount of clay, over half of the grains being of the size called "silt." It may appear quite cloddy but the lumps can be readily broken, and when pulverized it feels soft and floury. Either dry or moist it will form casts that can be handled freely without breaking. If squeezed between thumb and finger it will not "ribbon" but will give a broken appearance.

Clay loam.—A clay-loam soil in the field is dense and compact and breaks into clods or lumps. These are hard to break when dry. When the soil is moistened it can be pinched between the thumb and finger to form a thin "ribbon" which will break readily, barely sustaining its own weight. The moist soil is plastic and will form a cast that will bear much handling. It does not crumble readily but tends to work into a heavy puddled mass.

Clay.—A clay soil is dense and compact, forming very hard lumps or clods. It is composed of very fine particles, which stick together to make a very plastic or putty-like mass when wet. The soil can be pinched out to form a long, flexible "ribbon."

Adobe.—The term "adobe" indicates a structural condition. Any soil that will shrink on drying and break into blocks with wide cracks is called an adobe. Most adobes are clay adobes, but there are clay-loam adobes, and some sandy-loam adobes have been found.

All of the above classes of soil, if mixed with a considerable amount of gravel or stone, may be classed as gravelly sandy loams, gravelly clays, etc., or as stony sandy loams, etc. Sandy clay and sandy clay loams may also occur.

CLAY CONTENT A VITAL FACTOR

The importance of the Bureau of Soils bulletins lies to a large extent in the fact that they present accurate information concerning the clay content of the soil. From an agricultural standpoint this is a vital factor, because the amount and character of the clay present determine whether the soil is light and porous and of high productivity, or heavy, hard to drain, and difficult to work. To the highway engineer and road builder a knowledge of the quantity and character of the clay content is important because it seems to be a well established fact that these factors determine the degree to which the soil will heave when frozen or shrink and swell with variations in moisture content. The amount and kind of clay in the soil are, therefore, critical elements both to the agronomist and the highway engineer; and it is chiefly because they supply reliable information with regard to these elements that the maps made by the Bureau of Soils for agricultural purposes are an invaluable aid in making a road subgrade study.

It is true that these soil maps will not give information relative to all the soils which make up the subgrade. In rugged country the roadbed both in cuts and fills will be composed of soil which has been excavated from beneath the 6-foot layer included in the soil survey.[4] On the other hand, a large percentage of our highways are built through level or gently rolling topography and the excavations do not reach deeper than 6 feet. For these areas the soil maps are entirely adequate but even in the rugged country the soil surveys are of great assistance because the series names, such as Aiken, Melbourne, etc., are determined principally from the geological formation and the nature of the parent rock from which the soil was formed. As a rule the soils of any series, found at any depth (even greater than 6 feet) below the surface, bear a marked similarity. It follows therefore that for all conditions, the Bureau of Soils surveys, wherever they have been completed, present fundamental data from which the highway engineer may proceed to determine more specific and pertinent information concerning the subgrade.

Unfortunately soil surveys have not been completed and published for a considerable portion of the country and it becomes necessary, therefore, in many cases to identify the soils in the unmapped areas by comparison with similar samples taken from nearby areas which have been mapped or from mapped areas which bear the greatest resemblance to the unmapped area under consideration. Of course it is not necessary to attempt to determine the Bureau of Soils series and type name for a restricted subgrade study of, say, a 10-mile project, the results of which will not be used in further work. On the other hand, if it is expected to extend the subgrade studies over a large area or over an entire State highway system it is desirable to determine the Bureau of Soils nomenclature in order that the soil tests made upon one road project may be used for evaluating the quality of the same soil series and type when found on another project. It is not to be inferred from this that a single test of a soil type is considered sufficient to identify it as a good or bad subgrade wherever it is found. On the contrary there are found to be variations in the test values for the same soil series and type which make it necessary that a sufficient number of tests be made in any region so as to determine the upper and lower limits of the variations in the test values. In many cases these upper and lower limits will fall above the critical value which has been established for a good subgrade soil and therefore these will be classed thereafter as bad subgrade soils wherever they are found and without further tests. In the event that the upper and lower limits are found to be less than the critical value for a good subgrade soil they will be called good soils. In a number of cases, however, there will be found soils whose upper and lower test limits will fall above and below the critical value established for a region. These are doubtful soils and must be tested in all cases.

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[3] Professor of Soil Technology, University of California.

[4] The Bureau of Soils surveys formerly included mechanical analyses of the upper 6 feet of the soil in the western States and the upper 3 feet in the eastern States with often a brief description of the probable character of the subsoil at greater depths. The present practice is to describe all the soil and subsoil regardless of the depth down to the parent material with mechanical analyses. The depth now covered may reach to 10 or 15 feet. This makes the later surveys particularly valuable for evaluating highway subgrades since these data include all the soil encountered in making the cuts and fills.

DIGEST OF AVAILABLE DATA FIRST STEP IN SURVEY

In conducting a subgrade survey it is advisable as a first step to digest all available information concerning the soils of the area. If a Bureau of Soils survey bulletin is available the description of the soils contained in the bulletin may be reduced to a convenient tabular form as shown in Table 1.[5] From the same bulletin it will generally be possible to obtain a description of the topographic position of the various soils, i. e., whether they are found generally in mountainous, upland, or valley sections. With this information available it is convenient to prepare a diagram such as Figure 1, which, when used in the field, will aid the investigator by narrowing the range of soils to be considered in identifying the particular soils found on the project.

METHODS OF SURVEY VARIED TO MEET CONDITIONS

The methods used in making the subgrade survey will vary according to whether the road project is included in an area for which a recent and detailed Bureau of Soils bulletin is available, whether only a reconnaisance survey bulletin has been completed, or whether the area is unmapped. In the first case very little preliminary work is necessary before the field work begins. In the unmapped areas it is safer where possible to secure the assistance of a soil expert from a local agricultural college, because this is the most difficult problem that is likely to be encountered. The average case, however, may be said to be represented by an area covered by a reconnaissance survey because some of the old surveys were made before the modern methods and more detailed classifications were adopted and many of these old records are in existence. To illustrate a method which may be applied under average conditions the writer has selected as a basis for discussion the survey made on a 53-mile portion of the Pacific Highway in a section of the State of Washington covered by a Bureau of Soils reconnaissance survey.

FIELD INVESTIGATIONS

After a thorough study of the United States Bureau of Soils survey bulletin, the field work of the subgrade study was begun. A Dodge sedan automobile, a remnant of the World War, was used for the transportation of the investigators and the soil-testing apparatus. A closed car is particularly adapted to this type of work because it provides shelter for taking notes during rainy and stormy weather, which is the best time to observe drainage conditions. First the investigators rode or walked over the entire project and established the survey stations of the changes in soil types. An approximate idea of the location of the soil series was obtained from the course of the project on the Bureau of Soils map (Fig. 2); but the condensed and tabulated descriptions of the soils of the region, similar to those given in Table 1, and the diagram of their location with respect to the topography shown in Figure 1 furnished the most reliable means of checking the identification of the soil series. For instance, it was evident from Figure 1 that a residual soil found in the rough, mountainous districts must be either Olympic, Aiken, Melbourne, or Underwood. By a comparison of the field observations with the characteristics shown in Table 1 it was then possible to arrive at the series name by a process of elimination, checking the types by feeling between the fingers. The importance of this latter check can not be unduly stressed. It was possible by this means in the Pacific Highway study to locate in the vicinity of streams narrow belts of the Chehalis series which were not considered of sufficient importance by the Bureau of Soils to include in the reconnaissance survey map.

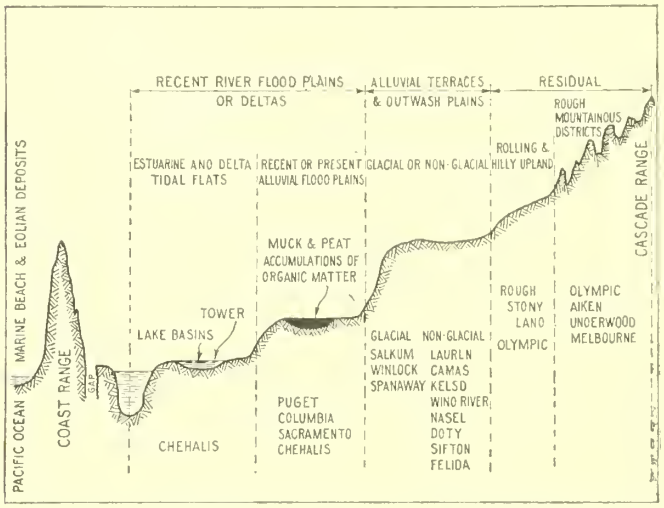

FIG. 1.—A general topographical cross section of southwestern Washington showing the location of the several soil series. Such diagrams greatly simplify the identification of soils by reducing the number of possibilities

The boundaries between the soil types were often well defined and frequently could be determined within 1 or 2 feet. At other places the transition from one soil type to another was gradual and it was necessary to approximate the point of change within a belt perhaps 40 or 50 feet wide. Having determined the points of change, the various series and types of soils encountered and the survey stations of the points of change were tabulated as shown by the example given in Table 2.

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[5] The data in Table 1 were digested from the Reconnaissance Survey of Southwestern Washington, U. S. Department of Agriculture, Bureau of Soils, issued May 15, 1913.

TABLE 1.—*Description and history of some of the soils encountered on the Pacific Highway*

Series	Colors		Derivation	Origin and material	Topography	Characteristics	Drainage	Types
	Soil	Subsoil						
RESIDUAL SOILS								
Olympic	Light brown to reddish brown.	Brown lighter than soils.	From basaltic rocks which underlie surface at from 3 to 25 feet and occur in weathered fragments on surface and in soil and subsoil.	Frequently of silty texture. In small areas some andesitic material and locally some colluvial material from the same rocks has been included.	Rough, hilly, or mountainous. Some areas comparatively level.	Outcrops of basalt and presence in soil of fragments of same rock.	Thorough to excessive.	Stony loam (Ost). Loam (Ol). Silt-loam (Osl). Stony clay-loam (Os). Silty clay-loam (Osc). Clay (Oc). Dark colored landslide phase.
Aiken	Dark brick red or purplish red to dark.	Bright red or reddish brown.	From basalt with admixture in some places of materials from andesitic or other eruptive rocks.	Residual bowlders and small fragments of rock found through soil mass. Bed rock lies shallow.	Areas lie on slopes or plateau-like uplands. Gently sloping to steep, rough, and broken.	Brick-red color of soil.	Efficiently to excessively drained.	Stony clay (As). Silty clay-loam (Ac).
Underwood	Yellow brown to gray brown.	Same as soil	Residual origin, but include some areas of colluvial formation.	Fragments of basalt size of small gravel to large bowlders, many of spherical form frequently found in soil and subsoil.	Rolling to steep and hilly foothill districts and foot slopes of mountains.	Numerous soft iron pellets occur in soil. Most numerous near surface and range from coarse sand to pellets ½ inch in diameter.	Well drained.	Loam (Ul).
SOILS OF THE GLACIAL PLAINS AND RIVER TERRACES								
Lauren	Reddish or light brown to dark brown.	Light brown or brown, sometimes mottled.	Probably formed from ancient alluvial or marine sediments derived from basaltic rocks, porous.	Soil underlain by porous substratum of stratified sand and fine gravel with occasional pockets or strata of coarse gravel.	Occupies elevated terraces lying adjacent to and above present stream valleys.	Presence of soft iron-cemented pellets on surface.	Well drained.	Gravelly coarse sandy loam (Lg). Sandy loam (Lsl). Fine sandy loam. (Lf). Silt-loam (Ls).
Camas	Light to dark brown.	Light to reddish brown.	Recent alluvial terraces from basaltic rocks.	Type occupies low terraces and alluvial fans at mouths of small creeks emptying into Columbia. Soil and subsoils contain rounded or subangular gravel and bowlders.	Surface level to gently sloping or undulating.	Soils contain numerous iron pellets. Outcrops of basalt and angular fragments of basalt occur on steeper slopes.	do	Stony gravelly loam (Cst). Gravelly sandy loam (Gg). Silt-loam (Cl).
Kelso	Grayish to reddish brown.	do	Material forming terraces which lie about 40 feet above bed of present flood plains is derived from glacial deposits and in part from areas occupied by residual Olympic and Melbourne soils.	Subsoil underlain by substratum of gray silt-clay and sand containing pockets of fine water-worn gravel. Basaltic bed rock.	Surface gently sloping or undulating.		do	Silty clay-loam (Ks).
SOILS OF THE RECENT FLOOD PLAINS AND DELTAS								
Sacramento	Dark gray to drab or black.	Drab or brown may be mottled.	Recent alluvial flood plain material deposited from shifting stream currents or slack flood waters. Has been transported long distances.	Comes from quartz-bearing and quartz-free igneous rock and metamorphic rock and some from sedimentary rock and glacial material. Contains a large amount of organic matter. Deeper subsoil at 3 to 5 feet is fine sand.	Occupies stream bottoms and flood plains. Often marked by sloughs subject to overflow. Surface varies from irregular to smooth.	Large amount of organic matter.	Subject to overflow.	Silty clay-loam (Sc).
Chehalis	Gray or drab to reddish brown, dark brown to black.	Yellow-gray, sometimes mottled, light brown, dark brown, or black.	Material has been eroded mainly from fine-textured soils occupying the uplands and deposited in river valleys in times of flood.	Recent alluvial, occupying valleys of streams that traverse humid regions of residual soils from basalt and sedimentary rocks.	Almost level. Sloughs or abandoned stream channels are frequent.	Heavier types contain much organic matter.	Good natural drainage.	Loam (Ch). Silty clay-loam (Ccl). Clay-loam (Cc). Silty clay (Cs). Clay (C).

TABLE 1.—*Description and history of some of the soils encountered on the Pacific Highway*—Continued

SOILS OF THE MARINE BEACH AND ÆOLIAN DEPOSITS

Series	Colors		Derivation	Origin and material	Topography	Characteristics	Drainage	Types
	Soil	Subsoil						
Westport fine sand.	Gray to white sand.		Shore drift	Material which has been built up above sea level by the combined action of waves and wind.	Occupy spits on each side of entrance to Willapa Bay and narrow strips on the seaward side of a part of the areas occupied by Westport fine sand.	No agricultural value. Lie above high tide. Inundated during heavy storms.	Good.	Fine sand (We).
Coastal beach and dune sand.	Gray to light yellow gray fine sand.	Mottled yellow, gray and brown fine sand.	Material derived from beach deposits. Type confined to western part of Pacific County, Wash.	Aeolian origin. Top, incoherent, fine sand, 12 to 14 inches. Subsoil the same to depth of 3 feet or more. Surface has small percentage of organic matter.	Marked by a series of low, rounded ridges 5 to 25 feet high with corresponding depressions between. Lies very little above sea level. A few small level areas occur.	Typical mottled subsoil. Vegetation on long sandy areas a stunted growth of fir. Dense under-growth of salal.	do	Costal beach and dune sand (Cd).

SOILS DERIVED MAINLY FROM ORGANIC DEPOSITS

Series	Soil	Subsoil	Derivation	Origin and material	Topography	Characteristics	Drainage	Types
Muck and peat	Brown	Grayish black to black.	Occupy poorly drained areas.	Occur in basins where conditions are favorable for growth and preservation of rank vegetation. Organic matter in various stages of decomposition. Consist of few inches of coarse vegetable fiber underlain by 18 to 20 inches of grayish black to black muck which in turn rests on a bed of peat. Underlain by fine sand at 3 to 5 feet.	Areas are flat and lie low. Covered with water during a part of the year. Some areas occur on terraces or uplands and are underlain by impervious clay.	Underlying sand at 3 to 5 feet.	Poorly drained.	Muck and peat (Mp).

TABLE 2.—*Identification of soil types*

Town	Mile number	Station to station		Soil series	Soil type	Abbreviation
La Center	0	850				
		879+23-	902+80	Felida	Silt-loam	Fs.
	1	902+80-	933+00	do	do	Fs.
		933+00-	937+00	Aiken	Silty clay-loam	Ac.
		937+00-	949+00	Felida	Silt-loam	Fs.
		949+00-	955+60	Aiken	Stony clay	As.
	2	955+60-	1,008+40	do	do	As.
	3	1,008+40-	1,012+00	do	do	As.
		1,012+00-	1,045+50	Felida	Silt-loam	Fs.
		1,045+50-	1,058+50	Aiken	Stony clay	As.
		1,058+50-	1,061+20	Felida	Silt-loam	Fs.
	4	1,061+20-	1,089+00	do	do	Fs.
		1,089+00-	1,114+00	Aiken	Stony clay	As.
	5	1,114+00-	1,120+00	do	do	As.
		1,120+00-	1,124+00	Puget	Fine sandy loam.	Pl.
		1,124+00-	1,125+50	Felida	Silt-loam	Fs.
		1,125+50-	1,133+50	Puget	Fine sandy loam.	Pl.
		1,133+50-	1,135+00	Felida	Silt-loam	Fs.
		1,135+00-	1,138+00	Puget	Fine sandy loam.	Pl.
		1,138+00-	1,143+00	Felida	Silt-loam	Fs.
Bridge over North Fork Lewis River		1,143+00-	1,155+30	Bridge	None	
		1,155+30-	1,166+80	Puget	Fine sandy loam.	Pl.
Woodland	6	1,166+80-	1,179+40	do	do	Pl.
		1,179+40-	=84+32			
		84+32-	99+00	do	do	Pl.
		99+00-	100+35	do	Silt-loam	Ps.
		100+35-	=0+35			
		0+35-	23+00	do	do	Ps.
		23+00-	24+17	Kelso	Silty clay-loam	Ks.
	7	24+17-	51+00	do	do	Ks.
		51+00-	76+97	Olympic	do	Osc.
	[1] 8	76+97-	129+77	do	do	Osc.
	41	152+89-	164+00	Toutle	Very fine sand	Ts.
		164+00-	168+50	Olympic	Silty clay-loam	Osc.
		168+50-	172+00	Toutle	Very fine sand	Ts.
		172+00-	189+00	Toutle over. Olympic	Very fine sand over Silty clay-loam	Ts. Osc.
		189+00-	198+50	Puget	Fine sandy loam.	Pl.
Toutle River bridge		198+50-	200+00	Bridge	None	
		200+00-	201+50	Kelso	Silty clay-loam	Ks.
		201+50-	205+69	Toutle	Very fine sand	Ts.
	42	205+69-	223+00	do	do	Ts.
		223+00-	243+00	Puget	Silt-loam	Ps.
		243+00-	252+00	Kelso	Silty clay-loam	Ks.
		252+00-	258+49	Olympic	do	Osc.
	43	258+49-	270+00	do	do	Osc.
		270+00-	281+00	Puget	Silt-loam	Ps.
		281+00-	304+00	Kelso	Silty clay-loam	Ks.
		304+00-	311+29	Olympic	do	Osc.
	44	311+29-	348+00	do	do	Osc.
		348+00-	364+09	Kelso	do	Ks.
	45	364+09-	389+00	do	do	Ks.
		389+00-	=0+00			
		0+00-	27+89	do	do	Ks.
	46	27+89-	62+00	do	do	Ks.
		62+00-	80+69	Winlock	Silty clay	Wsc.
	47	80+69-	92+50	do	do	Wsc.
		92+50-	318+17			
		318+17-	277+18	do	do	Wsc.
	48	277+18-	224+38	do	do	Wsc.
	49	224+38-	171+58	do	do	Wsc.
	50	171+58-	118+78	do	do	Wsc.
	51	118+78-	79+00	do	do	Wsc.
		79+00-	72+00	Chehalis	Silty clay-loam	Ccl.
		72+00-	65+98	Salkum	do	Ss.
	52	65+98-	13+18	do	do	Ss.
	53	13+18-	11+15	do	do	Ss.
North end Toledo bridge		11+15-				

[1] Data for miles 9–40, inclusive, omitted for sake of brevity.

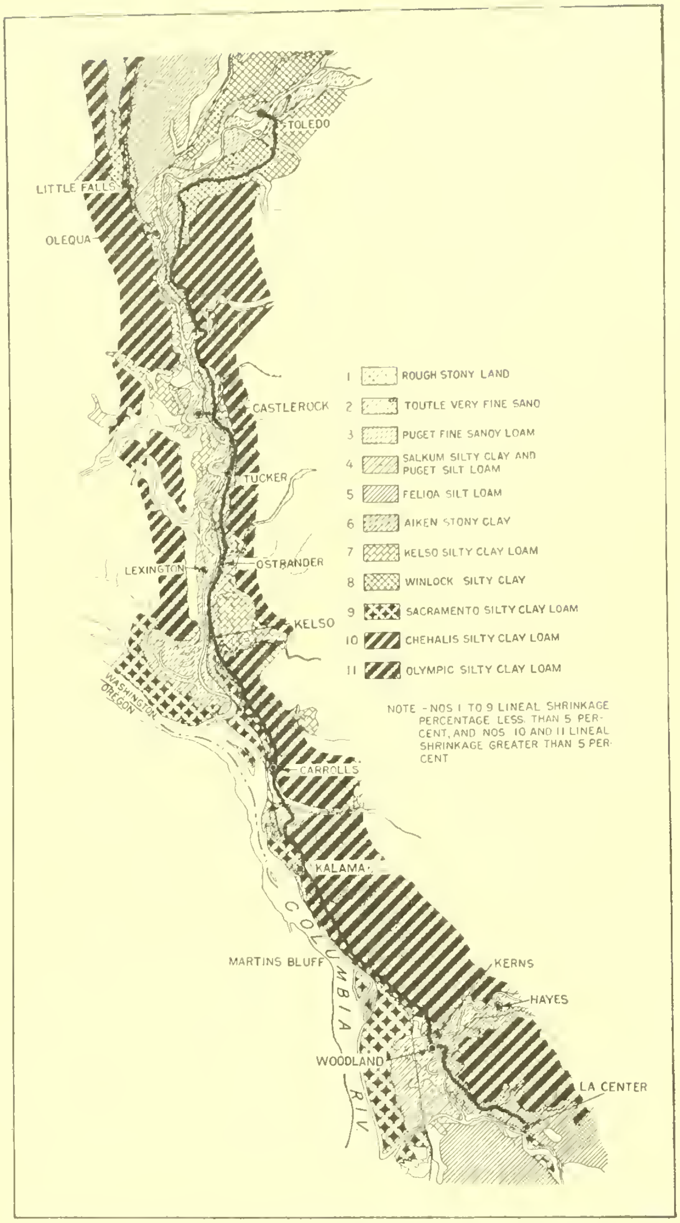

FIG. 2.—The variations in soil types on the Pacific Highway between La Center and Toledo, Wash., are shown by the legends which shade from white to black with the increase in the clay content of the soil

The project was then retraced and representative samples were selected from the types previously identified. These samples consisted of two full seamless galvanized salve cans of each type. The cans were 3 inches in diameter and 2½ inches deep. Some of the excavations being of such a depth that the subgrade was composed of material beneath the top layer described in the Bureau of Soils survey, special samples of these subsoils were taken and these were treated as special types.

TABLE 3.—*Results of field tests*

Sample No.	Location of sample: Layer	Location of sample: Station	Mile No.	Soil series and type	Moisture equivalent (average of 2 runs)	Lineal shrinkage (average of 2 runs)	Volumetric shrinkage (computed)
					Per cent	*Per cent*	*Per cent*
63	Topsoil	996+25	2.8	Aiken stony clay	31.2	2.49	7.3
64	Subsoil	996+25	2.8	do	35.9	9.19	25.0
65	Topsoil	1,080+50	4.4	Felida silt-loam	28.3	2.4	7.3
66	Subsoil	1,080+50	4.4	do	30.7	4.0	11.7
67	do	1,141+00	5.5	Locally known as blue clay.	22.8	2.9	8.7
68	do	174+00	9.8	Olympic clay	57.2	14.1	36.1
69	do	492+00	27.0	do	40.3	9.4	25.8
70	Topsoil	505+00	27.3	Puget fine sandy loam.	26.3	0.1	0.5
71	Subsoil	505+00	27.3	do	34.0	1.3	4.0
72	Topsoil	513+00	27.4	Sacramento silty clay-loam.	43.6	3.6	10.6
73	Subsoil	513+00	27.4	do	42.6	4.3	12.5
74	Topsoil	16+50	28.7	Kelso silty clay-loam	21.9	1.1	3.1
75	Subsoil	16+50	28.7	do	23.6	0.6	2.0
76	Topsoil	19+83	28.8	Puget silt-loam	37.3	2.1	6.1
77	Subsoil	19+83	28.8	do	27.8	1.6	4.8
78	Topsoil	12+00	29.3	do	36.8	1.0	3.0
79	Subsoil	12+00	29.3	do	26.5	0.5	1.5
80	Subsoil, 20 feet down.	141+00	31.8	Kelso silt-loam	26.8	0.5	1.5
81	Topsoil	236+00	33.6	Kelso silty clay-loam	34.4	3.2	9.8
82	Subsoil	236+00	33.6	do	26.9	1.6	5.0
83	do	105+00	40.1	Toutle gravelly coarse sand.	8.2	.0	.0
84	Topsoil	119+00	40.4	Toutle very fine sand.	26.4	.0	.0
85	Subsoil	119+00	40.4	do	32.6	0.6	2.0
86	Topsoil	139+22	40.7	Olympic silty clay-loam.	36.4	7.8	21.8
87	Subsoil	139+22	40.7	do	33.5	3.6	10.6
88	Topsoil	168+00	41.3	Winlock silty clay	36.4	3.5	10.5
89	Subsoil	168+00	41.3	do	30.1	4.4	12.6
90	Topsoil	73+50	46.9	Chehalis silty clay-loam.	41.3	8.1	22.0
91	Subsoil	73+50	46.9	do	35.3	7.1	19.8
92	do	25+00	52.8	Salkum silty clay	40.2	1.9	6.0
93	do	25+00	52.8	do	41.3	1.0	3.0

TABLE 4.—*Maximum and minimum values of the lineal shrinkage of soils on the Pacific Highway*

	Soil series and type	Abbreviation	Extent		Lineal shrinkage	
					Topsoil	Subsoil
			Miles	*Per cent*	*Per cent*	*Per cent*
Residual soils	Olympic silty clay-loam.	Osc	16.84	31.80	7.8	3.6
	Olympic clay	Oc	.90	1.70		9.4–14.1
	Aiken stony clay	As	2.03	3.83	2.5	9.2
	Aiken silty clay-loam	Ac	.08	.15		
	Rough, stony land	Rsl	2.65	5.00		
Soils of glacial plains and terraces.	Toutle gravelly coarse sand.	Tg	.05	.09		
	Toutle very fine sand	Ts	2.05	3.87		.6
	Salkum silty clay	Ss	1.15	2.17	1.9	1.0
	Winlock silty clay	Wsc	5.11	9.63	3.5	4.4
	Kelso silty clay-loam	Ks	10.84	20.44	1.1–3.2	.6–1.6
	Felida silt-loam	Fs	2.61	4.91	2.4	4.0
Soils of the recent plains and deltas.	Puget fine sandy loam	Pl	3.97	7.50	.1	1.3
	Puget silt-loam	Ps	2.11	3.97	1.0–2.1	.5–1.6
	Columbia fine sand	Cfs	.42	.79		
	Sacramento silty clay-loam.	Sc	.63	1.19	3.6	4.3
	Chehalis silty clay-loam.	Ccl	.13	.24	8.1	7.1
	Bridges, etc		1.44	2.72		
	Total		53.01	100.00		

By utilizing in this way the Bureau of Soils data in combination with the test by feeling with the fingers, it was possible to identify on the Pacific Highway project some 76 changes in the soil types, but these included only 16 different series and types and it was necessary to make field tests on only 31 samples in order to secure the limiting values for all the soils encountered. This gives some idea of the saving in time and labor made possible by the use of the Bureau of Soils bulletin. If this bulletin or similar data had not been available for the area it might have been necessary to make hundreds of tests in order to secure accurate information with regard to the soils of the entire project.

TABLE 5.—*Mechanical analysis of soil types found along the Pacific Highway by the United States Bureau of Soils survey*

Soil		Description	Fine gravel	Coarse sand	Medium sand	Fine sand	Very fine sand	Silt	Clay
Series	Type								
			P.ct.	*P.ct.*	*P.ct*	*P.ct.*	*P.ct.*	*P.ct.*	*P.ct.*
Olympic	Silty clay-loam.	Soil	0.5	3.0	1.4	5.6	10.0	53.6	25.7
		Subsoil	.3	2.0	1.5	4.2	6.5	44.6	40.7
Do	Clay [1]								
Aiken	Stony clay	Soil	.5	2.1	2.1	4.9	5.5	38.4	46.5
		Subsoil	.2	1.5	1.1	3.9	6.0	49.1	38.1
Do	Silty clay-loam.	Soil	.2	1.4	2.0	5.9	12.3	51.5	26.7
		Subsoil	.4	1.3	1.4	4.7	9.0	62.5	20.7
Rough, stony land.[2]									
Toutle	Gravelly coarse sand.	Soil	11.2	22.8	13.3	22.1	11.4	14.6	4.7
		Subsoil	12.1	22.4	12.7	22.3	12.5	14.4	3.5
Do.[3]	Very fine sand.								
Salkum	Silty clay	Soil	.5	1.2	1.1	3.3	3.1	59.4	31.5
		Subsoil	.2	1.1	1.2	3.2	3.6	55.6	34.6
Winlock	do	Soil	.0	1.4	.8	1.9	3.0	60.3	32.2
		Subsoil	.0	2.1	1.3	3.3	4.7	57.6	30.6
Kelso	Silty clay-loam.	Soil	.5	2.6	1.7	2.9	3.6	60.1	28.5
		Subsoil	.2	1.5	1.1	2.1	2.0	71.1	21.8
Felida	Silt loam	Soil	.4	3.2	1.7	3.3	11.9	63.6	15.8
		Subsoil	.0	1.0	.6	2.2	10.7	69.7	15.7
Puget	Fine sandy loam.	Soil	.4	.3	1.8	24.0	20.3	43.1	10.0
		Subsoil	.1	.3	3.1	66.7	21.0	6.7	1.7
Do	Silt-loam	Soil	.0	.2	.1	2.0	4.4	82.7	10.5
		Subsoil	.1	.4	3.5	37.4	22.9	32.5	3.7
Columbia [4]	Fine sand								
Sacramento	Silty clay-loam.	Soil	.0	.3	.6	1.7	.8	72.2	24.1
		Subsoil	.1	.5	.6	1.1	1.3	70.3	25.9
Chehalis	do	Soil	.0	.1	.1	2.6	5.8	64.8	26.4
		Subsoil	.0	.0	.2	1.9	1.0	64.7	32.1

[1] Large proportion of clay present.

[2] No analysis is given but considerable quantities of reddish brown clay are intermingled with the rock.

[3] No analysis given but a large proportion of very fine sand and a small percentage of organic matter is present in the soil. Large proportion of very fine sand in the subsoil.

[4] Practically all sand.

IDENTIFICATION OF BUREAU OF SOILS TYPE NAMES MAKES POSSIBLE A GREATER SAVING OF TIME ON FUTURE WORK

More important, however, than the saving of time on a single project is the fact that the correlation of the results of the tests with the series to which the soils belong will be useful in still further reducing the amount of testing required when the same soils are encountered on other projects. When the tests have been made on a sufficient number of roads to establish the limiting values for all the soils of a region, then it should be possible to make future subgrade surveys with a Bureau of Soils bulletin and map without making any tests except those which will always be required on the soils which vary in characteristics so as to be always doubtful.

The degree of certainty with which the characteristics of particular soils may ultimately be established by means of repeated tests may be appreciated by reference to Table 3. Two of the soils listed in the table—the Kelso silty clay-loam and the Olympic clay subsoil—will be used as an example. The lineal shrinkage percentages of the Kelso soil as determined by the tests varied from 1.1 to 3.2. The corresponding percentages for the Olympic soil varied from 9.4 to 14.1. Although the range in the latter case may seem rather great, there is in both cases a sufficient similarity in the values to establish the first soil as a good subgrade material and the latter as a bad subgrade material.

SOIL SAMPLES SUBJECTED TO SIMPLE FIELD TESTS

The soil samples selected in the manner described above were subjected to simplified field tests to determine their moisture equivalent and lineal shrinkage percentages. The methods employed in making these tests have been described in detail in a previous issue of PUBLIC ROADS,[6] and these descriptions are repeated here briefly for convenient reference.

Moisture equivalent percentage test.—The test is made by taking a 500-gram sample of air-dried soil, breaking up the lumps, placing the sample in a bowl, adding water slowly from a burette, and mixing the water and soil until it reaches the consistency of putty and may be compacted with a spoon or spatula without any free water remaining on the surface. Water is then allowed to drop upon the smoothed surface as long as it is absorbed. Before the moisture equivalent percentage is reached the sample will absorb water readily but when the critical value is passed the surface will assume a wet, shiny appearance. The sample is then weighed and dried out at 105° C. and the percentage of water is calculated on the basis of the dry weight of the soil.

Lineal shrinkage percentage test.—A 300-gram sample of soil wetted to moisture equivalent percentage is packed in ½-inch layers in a galvanized iron mold (1 inch by 1 inch by 10 inches) with a wooden tamper (½ inch by 1½ inches by 14 inches). The bars thus formed, after being weighed, are pushed from the mold upon a porcelain plate and calipered and then oven-dried at 105° C. and calipered again. The difference in length computed as a percentage of the wet length of the bar is considered the lineal shrinkage percentage.

The lineal shrinkage rather than the volumetric shrinkage is considered the preferable test because the vertical distortion of the subgrade due to the volume changes in the soil is believed to have the greatest effect upon the stability of the pavement.

The results of the tests made on the samples taken on the Pacific Highway are shown in Table 3, and the maximum and minimum values of the lineal shrinkage percentage for each soil together with the extent of the occurrence in the road bed are given in Table 4. This particular test is thus summarized because it is the critical test for subgrade soils. The moisture equivalent test is valuable principally as an indication of what the lineal shrinkage may be, and a similar value attaches to the mechanical analyses of the Bureau of Soils, shown in Table 5, because of the more or less close relation of the moisture equivalent percentage and the clay content of the soil.

SHORT CUTS POSSIBLE ON RECONNAISSANCE SUBGRADE SURVEYS

In advance of the definite classification of the subgrade soils of a region by the methods suggested above, it is possible to avoid a considerable amount of testing on rough, reconnaissance surveys by accepting the clay content of the soils as shown by the mechanical analyses of the Bureau of Soils as a rough criterion of their value. Such rapid surveys may be necessary when only a limited amount of time is available for the subgrade study. From the examination of a large number of soils in the Pacific Northwest it has been found to be the general rule that when the Bureau of Soils mechanical analysis shows a percentage of clay of 20 or less the lineal shrinkage percentage will be less than 5 per cent, which is below the limiting safe percentage for a subgrade soil in the Pacific Northwest. For instance, consider the Felida silt-loam. According to the mechanical analyses made by the Bureau of Soils (Table 5) this soil has a clay content of 15.8 per cent. The lineal shrinkage was found by tests to be 2.4 per cent, which confirms the soil as a good subgrade material. (See Table 3.) Similarly it will be observed that all the soils listed in Table 5 as having a clay content of 20 per cent or less are shown in Table 3 to have safe lineal shrinkage values.

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[6] Practical Field Tests for Subgrade Soils, PUBLIC ROADS, vol. 5, No. 6, August, 1924.

On the other hand, there are a number of soils the clay content of which exceeds 20 per cent which also are shown by the shrinkage tests to fall within the safe limit, but such soils should always be considered of doubtful value until their suitability is definitely proved by shrinkage tests. For example, Table 5 shows the Winlock silty clay to have a clay content of 32.2 per cent. This was therefore a doubtful subgrade material until the lineal shrinkage percentage was determined to be 3.5 per cent, which established it as a good subgrade soil. Sometimes, however, the clay content by mechanical analysis is very deceptive. An example of this may be seen in the Chehalis silty clay-loam with a clay content according to Table 5 of 26.4 per cent. While this percentage exceeds the limiting value of 20, adopted as the rough criterion for the Pacific Northwest, it does not greatly exceed it, yet the shrinkage tests showed this material to have a lineal shrinkage percentage of 8.1, which far exceeds the 5 per cent believed to be the limit for a good subgrade soil.

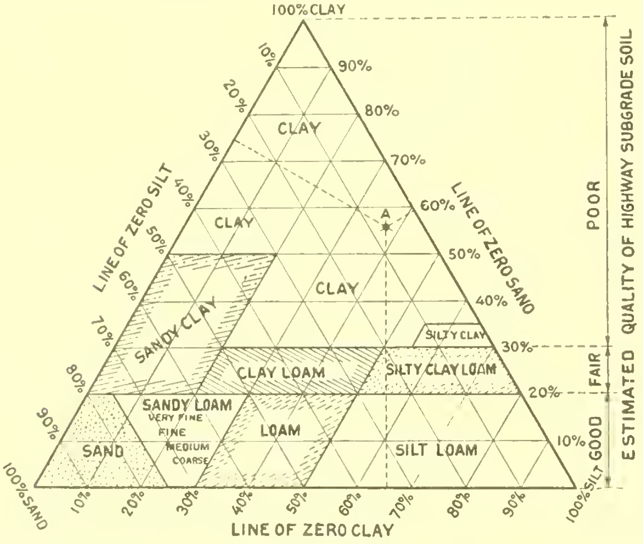

FIG. 3.—Trilinear soil classification chart for identification of good and bad subgrade soils in reconnoissance surveys. The nomenclature of the type areas is designed to agree with that of the United States Bureau of Soils

The writer does not advocate that the limits of clay content suggested for the Pacific Northwest be accepted for any region without verification. They are simply a guide to what seems to be a logical method. On the contrary, it is recommended that the various soil series and types as classified by the Bureau of Soils be tested a number of times over different areas of the same general region until it is believed that the upper and lower limits of the variations in the test values have been established. It is very likely that an investigator will then be able to walk over a road project through an area which has been mapped by the Bureau of Soils and, after identifying the soil types and determining their location, pick out at once the good and bad subgrades on the basis of tests made on similar soils found elsewhere in the region. But even under the most favorable circumstances doubtful soils may be found which will require testing before judgment may be passed upon them.

For rough reconnaissance survey purposes the soil classification triangle, Figure 3, may be used in connection with the Bureau of Soils survey bulletins for evaluating the subgrades. The triangle is plotted according to Professor Ferèt's method (Annales Des Ponts et Chaussées, 1892); and the nomenclature of the type areas is designed to agree with that adopted by the Bureau of Soils to describe the various soil types. The percentage of clay of 20 by mechanical analysis is shown as the upper limit for a good subgrade soil. Where the clay content by mechanical analysis is greater than or near 20 per cent the soil may make a doubtful or bad subgrade and these soils should be tested in all cases. Tests for lineal shrinkage show many soils within the zone of 20 to 30 per cent of clay to have a lineal shrinkage percentage of less than 5 per cent. These have therefore been roughly classified as fair subgrades. But, generally speaking, soils with a clay content by mechanical analysis of 30 per cent or more have a lineal shrinkage of more than 5 per cent and are, therefore, believed to be bad subgrade material.

In a former article[7] the writer stated that for road soil reconnaissance survey purposes the clay content by mechanical analysis may be considered as equal to the moisture equivalent percentage. This statement was based upon the following formula developed by Briggs and Shantz:[8] Moisture equivalent $= 0.02$ sand $+ 0.22$ silt $+ 1.05$ clay. Field tests have indicated that for rough reconnaissance survey purposes this formula could be modified so that the clay content could be considered as equivalent numerically to the moisture equivalent percentage. This is not always true, especially when the silt content is high and the clay content low. But because of the variable character of soils it is not believed that any formula of this kind can be developed which will hold true in all cases, nor even in a majority of cases. The moisture retentivity varies not only with the size of the soil particles but also with the character of the surface and structure of the grains, whether they be glazed or rough, solid or porous. In general, however, the moisture equivalent percentage does increase with the percentage of clay, and on the basis of tests made by the writer the clay content alone seems to be as good an indicator of the moisture equivalent as the formula of Briggs and Shantz.

INTERPRETATION OF THE SOIL TESTS

As previously pointed out the most important of the test results are the lineal shrinkage percentages. These afford the best indication of the probable behavior of the soil in the subgrade. But it must be borne in mind that the shrinkage percentage is materially affected by the presence of coarse material. The shrinkage test is made on samples of the soil from which all particles larger than 1 millimeter ($\frac{1}{25}$ inch) have been excluded by screening. If, therefore, a considerable amount of coarse material is present in the soil as it occurs in the field the actual field shrinkage may be considerably less than the test values indicate. Consequently it is necessary to estimate the amount of coarse material present in the field and to use this information in conjunction with the test values for shrinkage to identify finally the unsuitable subgrade soils.

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[7] Practical Field Tests for Subgrade Soils. Public Roads, Vol. 5, No. 6, August 1924.

[8] The wilting coefficient for different plants and its indirect determination. U. S. Dept. of Agriculture Bulletin No. 230.

TABLE 6.—*The reduction in the lineal shrinkage percentage of a Cove clay soil caused by the addition of varying amounts of sand of various gradings*

Kind of mixture	Moisture equivalent—average of two runs	Lineal shrinkage—average of two runs	Volumetric shrinkage (computed)
	Per cent	*Per cent*	*Per cent*
1 part medium sand to 1 part of soil	20.1	1.64	5.0
1 part medium sand to 2 parts of soil	25.7	2.75	8.0
1 part medium sand to 5 parts of soil	35.4	5.30	15.0
1 part medium sand to 10 parts of soil	38.1	9.08	24.9
1 part medium sand to 20 parts of soil	39.4	11.10	29.9
1 part coarse sand to 2 parts of soil	25.9	5.20	15.0
1 part fine sand to 2 parts of soil	27.5	7.32	20.5
All soil	40.5	13.46	35.2

	Per cent
Coarse sand:	
Passing ¼-inch and retained on 10-mesh sieve	33
Passing 20-mesh and retained on 40-mesh sieve	67
Medium sand:	
Passing 10-mesh and retained on 20-mesh sieve	33
Passing 20-mesh and retained on 40-mesh sieve	67
Fine sand:	
Passing 40-mesh and retained on 200-mesh sieve	100

TABLE 7.—*Locations on project where lineal shrinkage percentage of soil exceeds 5 per cent*

Locality	Station to station	Soil series
La Center to Woodland	933 - 937	Aiken.
	949 -1,012	Do.
	1,045+50-1,058+50	Do.
	1,089 -1,120	Do.
Woodland to Martins Bluff	51 - 219	Olympic.
Martins Bluff to Kalama	219 - 452	Do.
Kalama to Carrolls	474+50- 589+07	Do.
	589+07- 0+00	
	0 - 67+50	Do.
	78+50- 80+50	Do.
	230 - 250	Do.
Carrolls to Kelso	250 - 319+50	Do.
	322+50- 509	Do.
Kelso to Ostrander	None	
Ostrander to Castlerock	None	
Castlerock to Toledo	127 - 148	Do.
	164 - 168+50	Do.
	172 - 189	Do.
	252 - 270	Do.
	304 - 348	Do.
	72 - 79	Chehalis.

TABLE 8.—*Locations on project where lineal shrinkage of soil together with intermingled rock is estimated to exceed 5 per cent*

Locality	Station to station	Soil series
La Center to Woodland	933 - 937	Aiken.
	1,045+50-1,058+50	Do.
	1,089 -1,120	Do.
Woodland to Martins Bluff	162+50- 185	Olympic.
	202 - 207+50	Do.
Martins Bluff to Kalama	281 - 313	Do.
	314+50- 452	Do.
Kalama to Carrolls	12 - 33	Do.
	230 - 250	Do.
Carrolls to Kelso	250 - 255	Do.
	346 - 381	Do.
	390 - 421	Do.
	449 - 498+50	Do.
Castlerock to Toledo	127 - 148	Do.
	164 - 168+50	Do.
	304 - 348	Do.
	72 - 79	Chehalis.

The sample of Olympic clay taken at Station 174 in the sliding ground 4 miles north of Woodland on the Pacific Highway may be cited as an example to show the difference between the actual field shrinkage and the test shrinkage value. The sample passed through the 1-millimeter screen showed a lineal shrinkage of 14.1 per cent (a singularly high value). A sample of unscreened soil from exactly the same location, with some sand and small pebbles present, showed a lineal shrinkage of only 8.2 per cent, thus showing how materially the shrinkage is reduced by relatively small quantities of coarse material.

The effect of various quantities of coarse material on the lineal shrinkage is further shown by field tests made on a sample of Cove clay in Oregon. The results of adding to this soil various percentages of coarse, medium, and fine sand are shown in Table 6 and Figure 4. It will be observed that this soil when tested with the coarse material excluded showed a lineal shrinkage of 13.46 per cent, which would class it as a very undesirable subgrade soil. Yet the addition of 1 part of medium sand to 5 parts of the screened soil is sufficient to reduce the shrinkage to practically 5 per cent and thus convert it into a fairly satisfactory material.

On the Pacific Highway the tests showed the soil to have a lineal shrinkage percentage greater than 5 per cent at the locations listed in Table 7. But within these sections an examination of the soil in the field showed that there were considerable areas in which the soil was only a fraction of the mantle rock. In these cases the actual shrinkage of the soil in the field was estimated from the test values and the observed field conditions, and it was finally concluded that the lineal shrinkage would probably exceed 5 per cent only in the sections listed in Table 8, the aggregate length of which was 48,350 feet or 9.2 miles, and only about 17 per cent of the 53-mile section. The soils in these sections, it will be noted, were mainly of the Aiken and Olympic series, in which the clay content, according to Table 5, is higher than in any other soils found on the project.

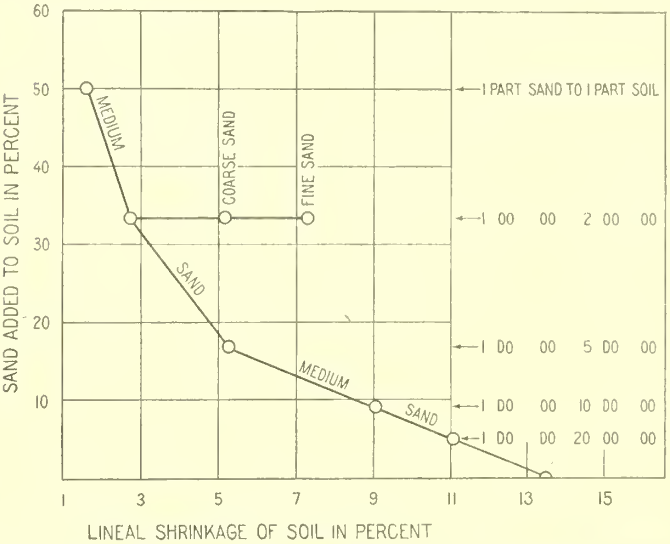

FIG. 4.—Results of tests made on a sample of Cove clay soil to show the decrease in lineal shrinkage resulting from the addition of sand

In making the above correction for the presence of coarse material in the soil the relative percentages of soil and rock were estimated and the resulting shrinkage was computed with the voids in the rock assumed at 50 per cent. For example, if the soil were found to have a lineal shrinkage of 8 per cent, and the mantle rock were composed of 70 per cent soil and 30 per cent rock with the soil filling the voids in the rock, it was computed that 55 per cent of the entire mass would shrink and the composite lineal shrinkage of the mass would be 4.4 per cent.

(Continued on page 115)

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EARTH PRESSURE AGAINST ABUTMENT WALLS MEASURED WITH SOIL PRESSURE CELLS

BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by J. V. McNARY, Highway Bridge Engineer, U. S. Bureau of Public Roads

THE different theories for determining the pressure of filling material against retaining walls now in general use give, for the same assumptions, comparable results. Variation in results is caused mainly by variation in the values assigned to the constants of the formulae. Different authorities give constants for the same conditions of filling which show a wide range. A digest of the standard specifications of the several States, which are presumed to cover what might be termed average conditions, shows that formulae are employed which give results that range from the pressure that would be caused by a fluid weighing 15 pounds per cubic foot to that which would be caused by a fluid weighing 36 pounds per cubic foot.

These variations emphasized by the not infrequent incipient failure and the occasional complete failure of abutment and wing walls have given rise to the feeling that in many instances the constants are not properly chosen or that other factors which receive insufficient attention are of major importance.

The soil pressure cell,[1] developed by the Bureau of Public Roads, has made possible the measurement of the force exerted by a fill against an abutment or retaining wall under actual working conditions. Tests have been made with this instrument at the Bennings Road and Sixteenth Street bridges in Washington, D. C., on hydraulic fills of clay at the Arlington Experimental Farm, Arlington, Va., and at the Skellit Fork bridge, near Wayne City, Ill. Only the Sixteenth Street and Skellit Fork bridge tests are described here, but a single reading from each of the tests mentioned above with formulae from several handbooks, all reduced to equivalent fluid pressures, are shown in Figure 1 for purposes of comparison.

The conclusion drawn from these actual tests is that the pressure of earth against a retaining wall should not under ordinary conditions be assumed to be less than that which would be developed by a fluid weighing 30 pounds per cubic foot. By definitely showing that a direct relation exists between the high pressures measured and a condition of high moisture content prevailing at the time, the tests also indicate the importance of suitable provisions for draining the fill. Such provisions should be regarded as major features of the work of equal importance with the design and construction of the wall itself.

THE SIXTEENTH STREET BRIDGE TEST

The Sixteenth Street Bridge carries the street over Military Road. Military Road leaves the floor of the main valley and ascends to high ground to the east through a branch ravine and Sixteenth Street, descending at a uniform grade of about 5 per cent, from the high ground 1,500 feet to the south crosses this ravine and reaches the main valley floor about 1,000 feet north of the bridge site. Thus the bridge is located on a hillside and the topography is such that the south or uphill abutment and wing walls intercept the ground water flow and the surface drainage from the long hill above. The abutments are founded on good shale and

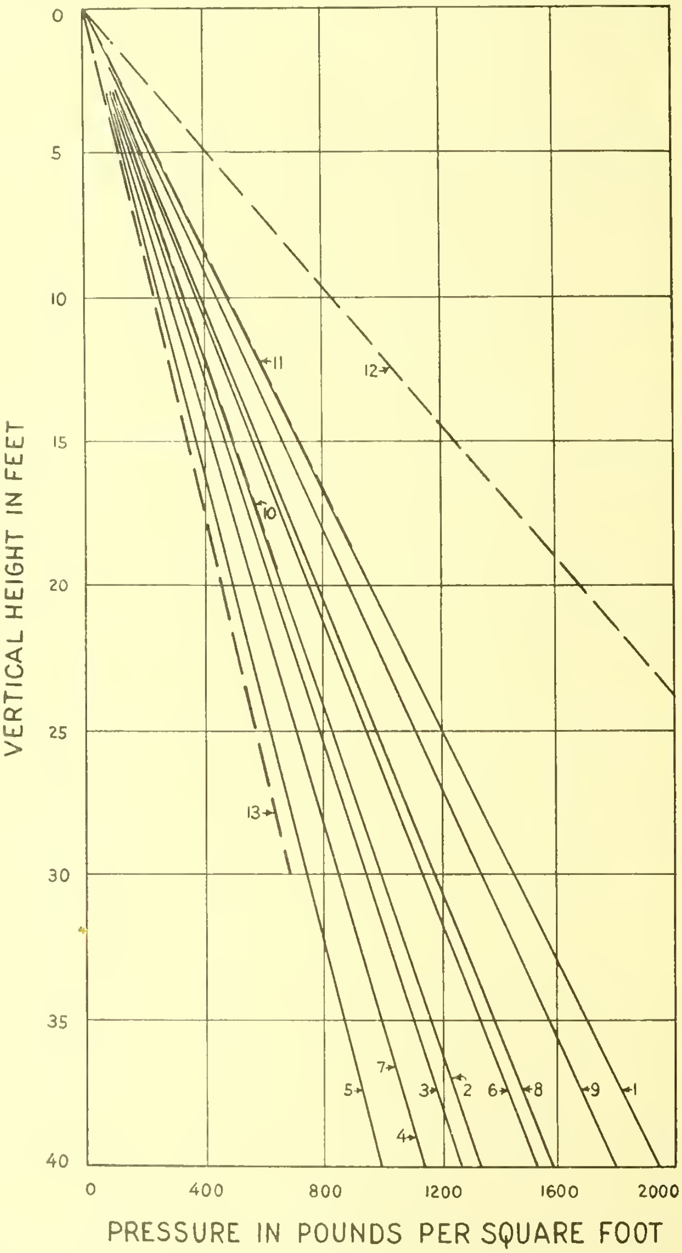

Fig. 1.—Pressures determined by Rankines formula for different kinds and conditions of filling compared with pressures determined by Bureau of Public Roads Soil Pressure cells. Results from Rankines formula shown by solid lines and dash lines show experimental results.

KEY

(1) Worst case of "ordinary earth, wet; weight per cubic foot, 120 pounds; angle of repose, 25°." Equivalent to fluid weighing 48.7 pounds per cubic foot. Hool and Johnson Concrete Engineers Hand Book.

(2) Most favorable case of "ordinary earth, wet; weight per cubic foot, 100 pounds; angle of repose, 30°." Equivalent to fluid weighing 33⅓ pounds per cubic foot. Hool and Johnson Concrete Engineers Hand Book.

(3) Retaining wall fill, weighing 100 pounds per cubic foot; angle of repose, 30°. Equivalent to fluid weighing 33⅓ pounds per cubic foot. Author gives C=16. Equivalent to fluid weighing 32 pounds per cubic foot. Paaswell.

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[1] For description of the soil pressure cell, see PUBLIC ROADS, vol. 3, No. 28, August, 1920.

(4) Customary assumptions; fill weighing 100 pounds per cubic foot; angle of repose, 1½ to 1=33° 40′. Equivalent to fluid weighing 28.7 pounds per cubic foot.

(5) "Dry earth; weight per cubic foot, 100 pounds; angle of repose, 36° 53′." Equivalent to fluid weighing 25 pounds per cubic foot. American Civil Engineers Pocket Book.

(6) "Mud; weight per cubic foot, 100 pounds; angle of repose, 26° 34′." Equivalent to fluid weighing 38.2 pounds per cubic foot. American Civil Engineers Pocket Book.

(7) "Sand, gravel, and clay; wet; weight per cubic foot, 115 pounds; angle of repose, 36° 53′." Equivalent to fluid weighing 28.7 pounds per cubic foot. American Civil Engineers Pocket Book.

(8) "Soil dumped into water; weight per cubic foot, 70 pounds, angle of repose, 15° 57′." Equivalent to fluid weighing 39.5 pounds per cubic foot. American Civil Engineers Pocket Book.

(9) "Clay dumped into water; weight per cubic foot, 80 pounds; angle of repose, 15° 57′." Equivalent to fluid weighing 45 pounds per cubic foot. American Civil Engineers Pocket Book.

(10) Bureau of Public Roads tests, Benning Road Bridge, D. C. Equivalent to fluid weighing 33.4 pounds per cubic foot.

(11) Bureau of Public Roads tests, Sixteenth Street Bridge, D. C. Equivalent to fluid weighing 48.4 pounds per cubic foot.

(12) Bureau of Public Roads tests on hydraulic fill of clay. Equivalent to fluid weighing 84.5 pounds per cubic foot.

(13) Bureau of Public Roads tests, Skellit Fork Bridge, Wayne City, Ill.; low moisture content in filling material; no rain between time of filling and date of readings. Equivalent to fluid weighing 23.2 pounds per cubic foot.

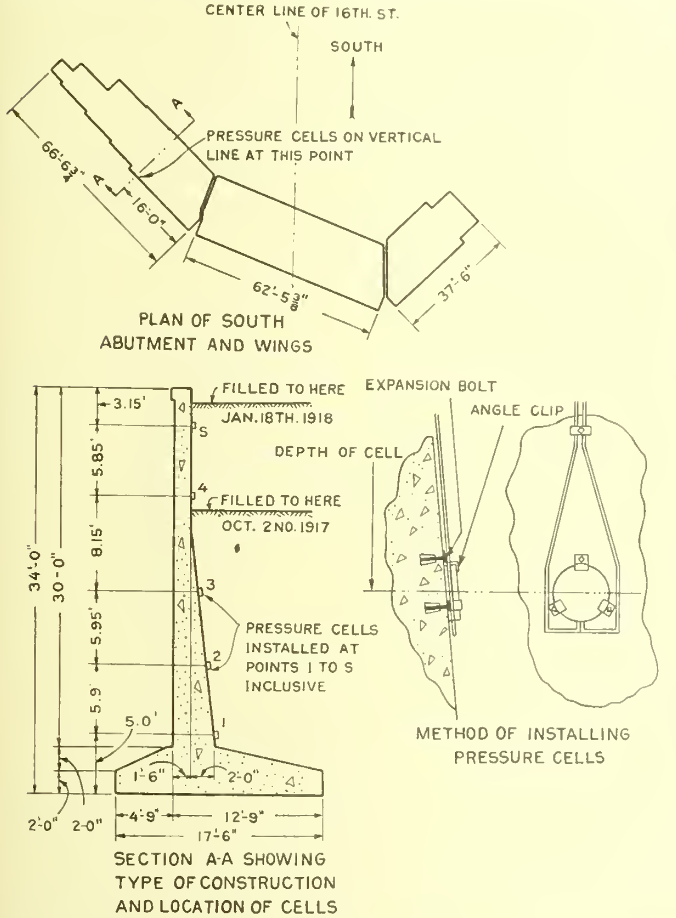

CENTER LINE OF 16TH. ST.
SOUTH
A
PRESSURE CELLS ON VERTICAL LINE AT THIS POINT
66'-6¾"
A
16'-0"
62'-5⅜"
37'-6"
PLAN OF SOUTH ABUTMENT AND WINGS
FILLED TO HERE JAN. 18TH. 1918
3.15'
5
5.85'
4
FILLED TO HERE OCT. 2ND. 1917
8.15'
34'-0"
30'-0"
3
5.95'
2
PRESSURE CELLS INSTALLED AT POINTS 1 TO 5 INCLUSIVE
5.9
1
5.0'
1'-6"
2'-0"
2'-0"
2'-0"
4'-9"
12'-9"
17'-6"
SECTION A-A SHOWING TYPE OF CONSTRUCTION AND LOCATION OF CELLS
EXPANSION BOLT
ANGLE CLIP
DEPTH OF CELL
METHOD OF INSTALLING PRESSURE CELLS

Fig. 2.—The plan of an abutment and a section of the wing wall of the Sixteenth Street Bridge

are constructed in three separate units. The breast walls or abutments proper are of mass concrete of the gravity type while the wings are of the reinforced-concrete, cantilever type. No provision for draining the fill was made in either the abutment or wings. The filling material which was placed with hand shovels, was very thoroughly compacted by a force of men with hand rammers. The shovelers and rammers were so proportioned that the material was well tamped in horizontal layers of from 1 to 3 inches in thickness. Filling was started August 29, 1917, and by October 2, 1917, had been carried up to a point 6.85 feet above cell No. 3 where work was discontinued. It was resumed the first of the year and filling was completed by January 18, 1918. Figure 2 shows the plan of this abutment and a section of the wing wall.

Five standard soil-pressure cells were placed back of the southeast wing wall. As the installation was made after the wall was completed it was not possible to set the faces of the cells in the plane of the back of the wall; instead the cells and their connections had to be attached to the back of the wall with expansion bolts. The location of the cells and the method of installing them are also shown in Figure 2.

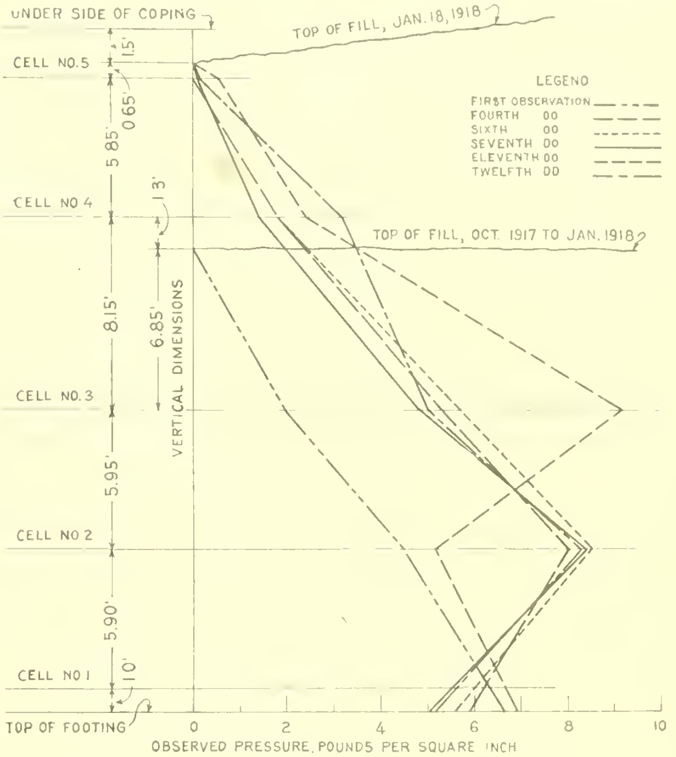


Fig. 3.—The observed pressures on the soil pressure cells used in the Sixteenth Street Bridge test

From October 2, 1917 to January 27, 1919, 12 observations were made at this bridge, the first 3 during the interval while work was suspended. The pressures read on the soil pressure cells are given in the following table and are shown graphically in Figure 3.

Observation	Date	Pressures as determined by pressure cells				
		No. 1	No. 2	No. 3	No. 4	No. 5
		Pounds per square inch	*Pounds per square inch*	*Pounds per square inch*	*Pounds per square inch*	*Pounds per square inch*
	1917					
1	Oct. 2	6.3	4.5	1.95	----	----
2	Nov. 12	6.3	4.5	2.2	----	----
3	Dec. 4	6.3	[1] 4.2	2.1	----	----
	1918					
4	May 4	6.2	8.0	5.3	[2] 1.8	(3)
5	May 9	6.6	8.4	5.7	1.6	0.2
6	May 14	(4)	8.5	5.7	1.8	0.2
7	June 17	5.4	8.4	5.0	1.4	0.1
8	June 28	5.5	8.4	4.8	1.4	0.7
9	Aug. 8	5.9	8.9	4.9	(5)	0.1
10	Oct. 11	5.8	8.5	4.8	[6] 1.7	0.5
11	Dec. 20	6.65	5.15	9.15	(5) (6)	0.55
	1919					
12	Jan. 27	5.6	8.28	5.00	[7] 3.2	0.15

---

[1] Air pressure ran to about 5 pounds per square inch before a short circuit was discovered.

[2] Cell No. 4 slightly clogged.

[3] Two feet broken off of wire pipe.

[4] Air pressure exceeded the earth pressure before a short circuit was discovered. Readings a few minutes later showed only 1.5 pounds per square inch. Cell should return to normal after fill readjusts itself

[5] Wire pipe broken, pipe system partially clogged.

[6] Air pipe closed.

[7] Reading taken on wire pipe, pipe almost closed

The absence of any definite increase of pressure on cell No. 1 from the increased depth of fill is probably due to the internal friction of the filling material, since this cell is only one foot above the top of the concrete footing which slopes away from the back of the wall.

In the eleventh observation an unusually low pressure on cell No. 2 is accompanied by an unusually high pressure on cell No. 3. This is believed to be due to readjustment within the fill. It is probable that the fill in the region of cell No. 2 subsided as the result of natural consolidation while the upper material was partially supported by arch action against the abutment in the region of cell No. 3, thus relieving cell No. 2 of pressure while cell No. 3 was subjected to the increased pressure created by the arch action. The next observation gave a normal reading on cell No. 2 with a low reading on No. 3 and a high reading on cell No. 4, showing that in the process of consolidation the arching action was working toward the top of the fill and gradually disappearing.

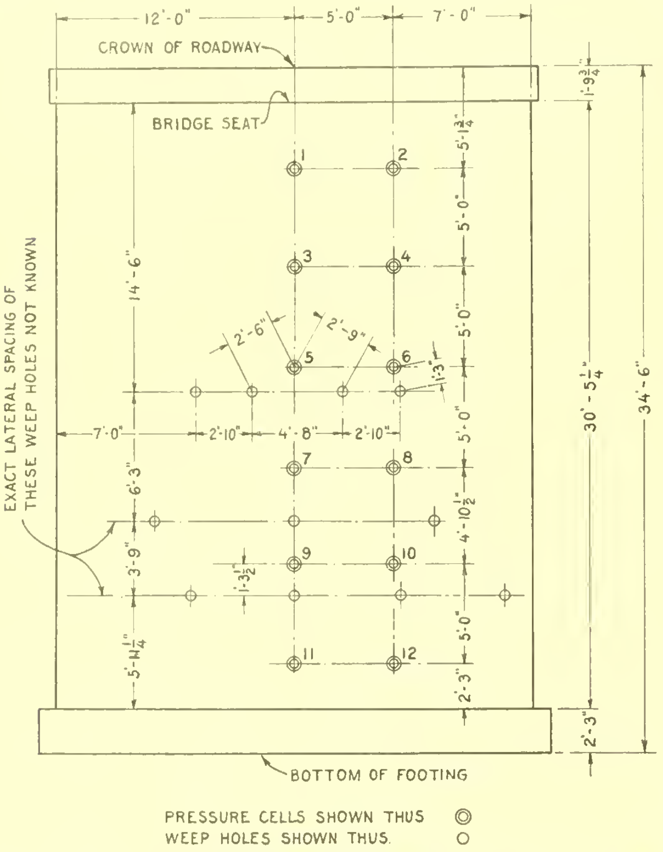


FIG. 4.—The location and spacing of the soil pressure cells used in the Skellit Fork Bridge test

The weight per cubic foot of a fluid that would give a total pressure equal to the total observed pressure for each observation is given in the table following:

Observation	Weight of equivalent fluid	Observation	Weight of equivalent fluid
	Pounds per cubic foot		*Pounds per cubic foot*
1	47.6	7	44.2
2	48.8	8	44.6
3	47.0	9	46.3
4	46.4	10	45.8
5	48.8	11	54.4
6	48.3	12	48.9

Although on good foundation material and of a cross-section that should satisfactorily support the ordinarily assumed loads, the wall has moved outward at the top about two inches, thus confirming the presence of the unusually high pressures shown by the pressure cells.

The filling material was of a clayey nature and the initial observation was undoubtedly affected by the thoroughness of the mechanical compacting while it was being placed. Continued high pressure, however, is most probably due to a high moisture content. As previously stated the wall is so situated that it intercepts both ground water flow and surface drainage and is not provided with any means of drainage. The joint between the wing wall and the abutment wall discharges seepage throughout its entire height showing that the fill becomes saturated to the top. Comparison with the Weather Bureau records shows that the higher pressures were observed after a period of several days precipitation and the lowest pressure at the end of a period of no precipitation. The highest pressure, shown by observation No. 11, was noted six days after a precipitation of 2.87 inches in the form of snow on unfrozen ground with the temperature above freezing each day.

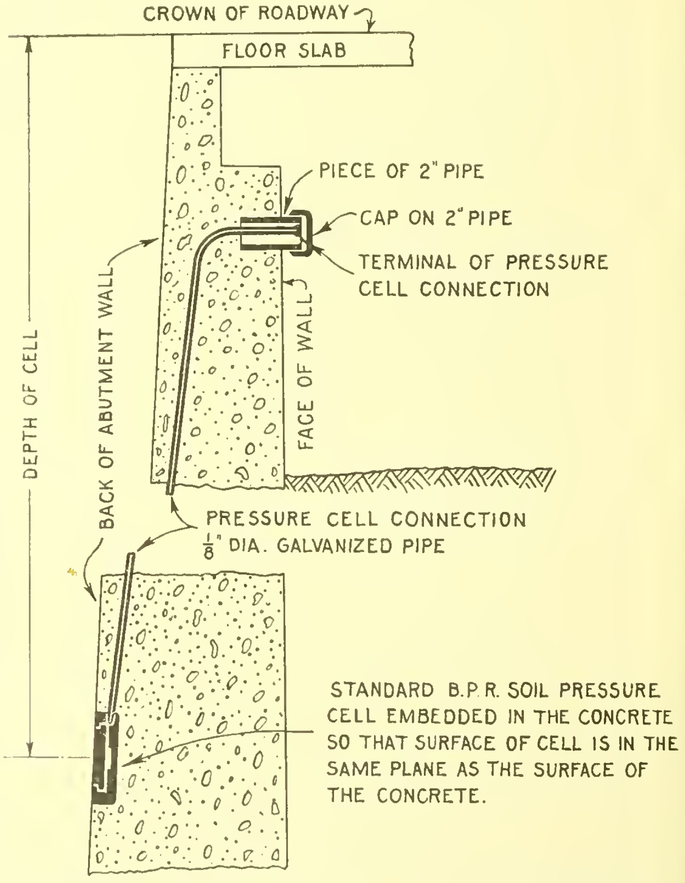

CROWN OF ROADWAY
FLOOR SLAB
PIECE OF 2" PIPE
CAP ON 2" PIPE
TERMINAL OF PRESSURE CELL CONNECTION
DEPTH OF CELL
BACK OF ABUTMENT WALL
FACE OF WALL
PRESSURE CELL CONNECTION ⅛" DIA. GALVANIZED PIPE
STANDARD B.P.R. SOIL PRESSURE CELL EMBEDDED IN THE CONCRETE SO THAT SURFACE OF CELL IS IN THE SAME PLANE AS THE SURFACE OF THE CONCRETE.

FIG. 5.—The manner of installing the soil pressure cells in the Skellit Fork Bridge test

SKELLIT FORK BRIDGE TEST

The test on the bridge over Skellit Fork was made by the Bureau of Public Roads in cooperation with the division of highways of the Illinois Department of Public Works and Buildings. The entire crossing consists of the bridge over the stream channel, and fills, on a level grade, raised a few feet above high water across the rest of the flood plain. The flood plain is subject to inundation practically every spring.

The abutment is of the Illinois standard reinforced U type, 32 feet, 3¼ inches from the top of the footing to the grade line, and 24 feet wide on the face wall.

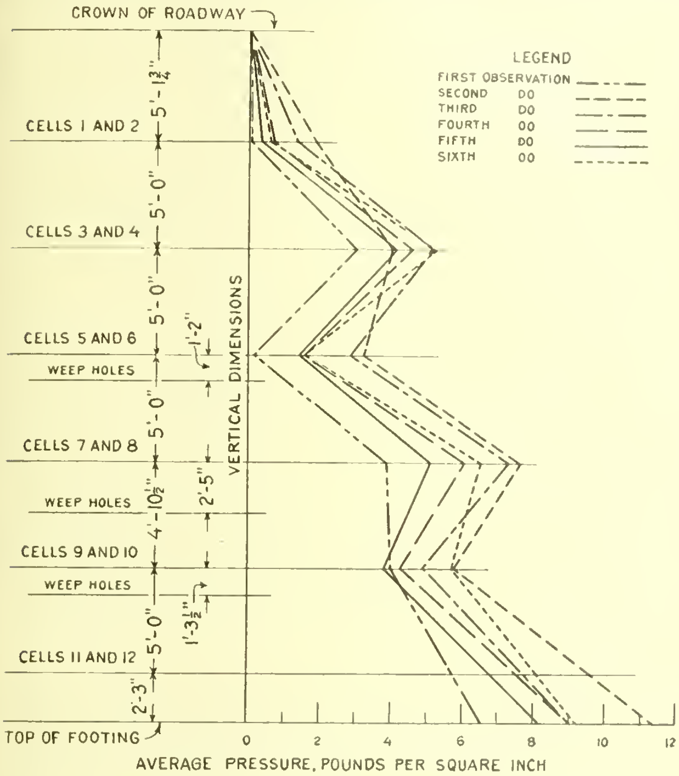


FIG. 6.—Average pressures on cells at the same elevation in the Skellit Fork Bridge test

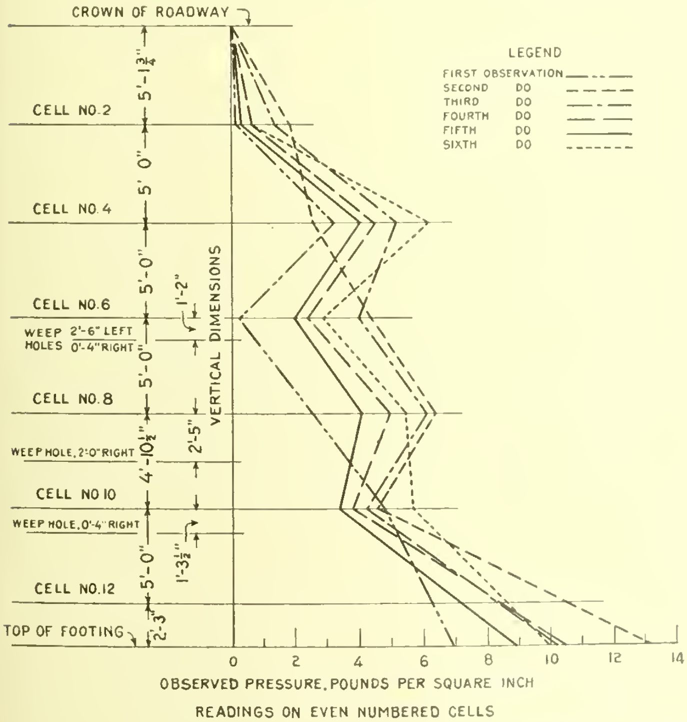


FIG. 7.—Observed pressures on the even-numbered cells in the Skellet Fork Bridge test

The Illinois standard practice in regard to methods of construction, drainage, and manner of placing the filling material were followed.

The installation was planned and the first cells placed by a representative of the Bureau of Public Roads after which the installation was completed by the Illinois division of highways. Two cells were used at each elevation in order to reduce the effects of local areas of high or low pressure. The location of the cells is shown in Figure 4 and the manner of installing them is shown in Figure 5. Observations were made jointly by representatives of the Bureau of Public Roads and the State division of highways. A total of six observations have been made up to the present time, the data from which are given in the following table:

Observation	Date	Pressures as determined by pressure cells (pounds per square inch)					
		No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
1	1921 Nov. 8	0.0	0.1	2.86	3.2	0.3	0.2
2	1922 Apr. 28	1.77	1.87	5.6	2.53	2.10	4.37
3	May 27	1.30	1.40	5.37	5.13	1.70	3.97
4	July 21	0.73	0.70	4.76	4.52	0.83	2.33
5	Aug. 28	0.50	0.27	4.28	4.05	0.90	1.93
6	1923 June —	0.57	0.70	4.50	6.15	0.50	2.35

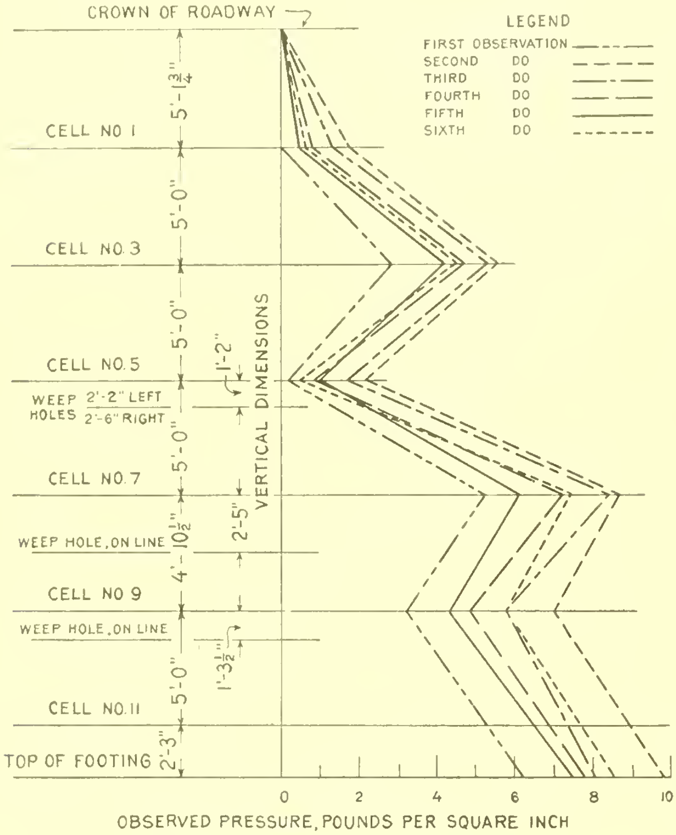
Observation	Date	Pressures as determined by pressure cells (pounds per square inch)					
		No. 7	No. 8	No. 9	No. 10	No. 11	No. 12
1	1921 Nov. 8	5.23	2.56	3.2	4.8	5.3	6.25
2	1922 Apr. 28	8.86	6.4	7.0	4.53	8.92	10.45
3	May 27	8.47	6.15	5.77	4.17	7.30	8.30
4	July 21	7.20	4.97	4.83	3.77	6.82	8.30
5	Aug. 28	6.15	4.08	4.33	3.33	6.50	7.15
6	1923 June —	7.50	5.55	5.75	5.70	7.70	8.63

The first observation was made November 8, 1921. The filling had been completed but a short time and there had been no rain between the time of completion and the making of the observations, so that the data represent the pressure due to an artificially compacted filling having a low moisture content. As the filling was done at the close of the dry season and considerable moisture would evaporate during the process of handling and placing of the material it is probable that the moisture content of the fill was lower than it might be expected to be at any subsequent time.

The second observation was made April 28, 1922, within 24 hours after high water had receded from the abutment. The spring of 1922 was marked by heavy snows, long continued heavy rains, and excessively high water, and the stream was over its banks a number of times during the month before the observation was made. The water content of the fill therefore was probably as near to the point of saturation as would ordinarily be obtained in a roadway embankment and abutment fill.

No unusual conditions existed at the time of the third, fourth, and fifth observations which were made in May, July, and August of 1922. The sixth observation was made during the first week of June, 1923, shortly after the subsidence of the spring floods and the moisture content of the fill was probably considerably above the average. The observed pressures are shown graphically in Figures 6, 7, and 8.

The readings on cells Nos. 5 and 6 are exceptionally low. An explanation of this phenomena might be found in the character and method of placing the fill in the vicinity of these cells, the possible interruption of the work for a considerable length of time near this elevation, and the weather conditions. However, data concerning these features are not available.



READINGS ON ODD NUMBERED CELLS

Fig. 8. Observed pressures on the odd-numbered cells in the Skellit Fork Bridge test

In the second observation cells Nos. 2 and 4 exhibit the same phenomena as cells Nos. 2 and 3 at the Sixteenth Street bridge in the eleventh observation. On the subsequent observations these cells showed normal readings.

It is believed that the pronounced irregularities of the pressure curves may be accounted for in part by the method of drainage. The weep holes consist of openings through the wall without any provisions for conducting the seepage to them. It would be expected therefore, that the portion of the fill drained would be immediately above the opening and its size would depend on the character of the filling material and lapse of time. Examination of the curves shows the peaks of pressure to occur at those cells most distant from the weep holes. The cells more distant from the weep holes show a greater increase in pressure when the moisture content of the fill is raised, and a greater reduction in pressure as the interval of time between the period of high moisture content and the time of observation increases.

The weight per cubic foot of a fluid that would give a total pressure equal to the total observed pressure for each observation is given in the following table:

Observation	Weight of equivalent fluid	Observation	Weight of equivalent fluid
	Pounds per cubic foot		*Pounds per cubic foot*
1	23.2	4	34.7
2	44.1	5	29.8
3	40.5	6	38.0

It will be noted that the lowest pressure was observed immediately after the fill was placed and was equal to that caused by a fluid weighing 23.2 pounds per cubic foot. The highest pressure occurred at the time of highest moisture content of the fill and was equal to that caused by a fluid weighing 44.1 pounds per cubic foot.

As the fill lost a part of its water content the pressures gradually decreased but did not go as low as the initial observation, although the fifth observation, which showed the next lowest pressure, was taken at the same time of year and when the fill, except for the solidification which had taken place during the year, was in practically the same condition. This pressure was equal to that caused by a fluid weighing 29.8 pounds per cubic foot.

Even after the fill had settled for a year and a half the pressure again became quite high when the moisture content increased, reaching a value equal to that which would be produced by a fluid weighing 38 pounds per cubic foot.

It is believed that the differences in pressure shown by the various readings are algebraic quantities made up of a gradual increase in pressure caused by the natural settling and compacting of the fill and an increase or decrease in pressure as the water content increases or decreases. The former factor will gradually decrease with the lapse of time so that after the fill has reached its final settlement the total variation in pressure may be attributed to the fluctuations in the water content of the filling material.

TRANSPORTATION OF MILK BY MOTOR TRUCK IN THE CHICAGO DAIRY DISTRICT

A STUDY OF HIGHWAY UTILIZATION BY THE U. S. BUREAU OF PUBLIC ROADS

Reported by E. L. BROWNE, Agricultural Economist

APPROXIMATELY 44,000 8-gallon cans of milk were delivered daily to the city of Chicago during 1924 by 26 railroads, which carried 68 per cent of the supply, and 141 motor trucks, which transported 32 per cent. If any milk consumed in the city was delivered in any other way the amount was negligible.

Thirty years ago the bulk of the supply was brought into the city in wagons and distributed by the individual producers. The city had then less than half its present population. In the course of the next year or two the growth of the population and the extension of urban development into the surrounding country crowded the dairy farms back beyond a wagon's haul distance of the city, and by 1898, according to an early report of the department of health, a large part of the city's supply was being delivered by railroad, the report showing a daily rail delivery by 17 roads of 16,100 8-gallon cans.

Another change in the method of transportation began about 1904, when the electric railroads began to take over a portion of the business from the steam roads. The amount hauled by the electric roads increased from year to year until the competition of the motor truck began to make itself felt, and finally the truck compelled the discontinuance of electric-road shipments in 1924. The movement reached its height between 1915 and 1920, and the largest electric carrier in the Chicago tributary area held on until January, 1924, although its shipments had decreased from 179,022 8-gallon cans in 1920 to 80,322 in 1923.

As shown by Table 1, the total daily delivery of milk to the city in 1910 was 31,245 8-gallon cans, of which 29,400, or 94 per cent, were transported by steam railroads; 600 cans, or 2 per cent, by electric railroads; and 1,245, or 4 per cent, by wagon. By 1924 the electric railroad and wagon shipments had ceased and only the steam railroads and the motor truck shared in the delivery of the 43,994 8-gallon cans, which were transported daily, the former with 68 and the latter with 32 per cent of the business. The motor truck had taken over all the business formerly handled by the electric roads and the wagons and a part of the business of the railroads, although the shipments of the latter remained practically the same in volume as they were in 1910.

TABLE 1.—*Milk delivered daily to Chicago consumers, 1910 and 1924*

Method of transportation	1910		1924	
	Number of 8-gallon cans	Per cent of total	Number of 8-gallon cans	Per cent of total
Steam railroads	29,400	94	29,904	68
Electric railroads	600	2		
Wagons	1,245	4		
Motor trucks			14,090	32
Total	31,245	100	43,994	100

REASONS FOR INCREASE IN MOTOR TRUCK DELIVERY

It is not difficult to account for the change from wagon to motor truck delivery. The building up of a circle of suburban towns, golf courses, and subdivisions about the city has simply crowded the dairy farms back until it is no longer possible to make deliveries by wagon on account of the excessive time required for the haul.

As to the other business that has been developed by the motor truck—a part of it comes from areas within a 50-mile radius of the city which formerly were too far distant both from the city and from the railroad shipping points to be reached by wagon haul. The balance has been taken from the steam and electric railroads, mainly within the 50-mile zone, as a result of certain advantages possessed by the truck, among which the following are the most important:

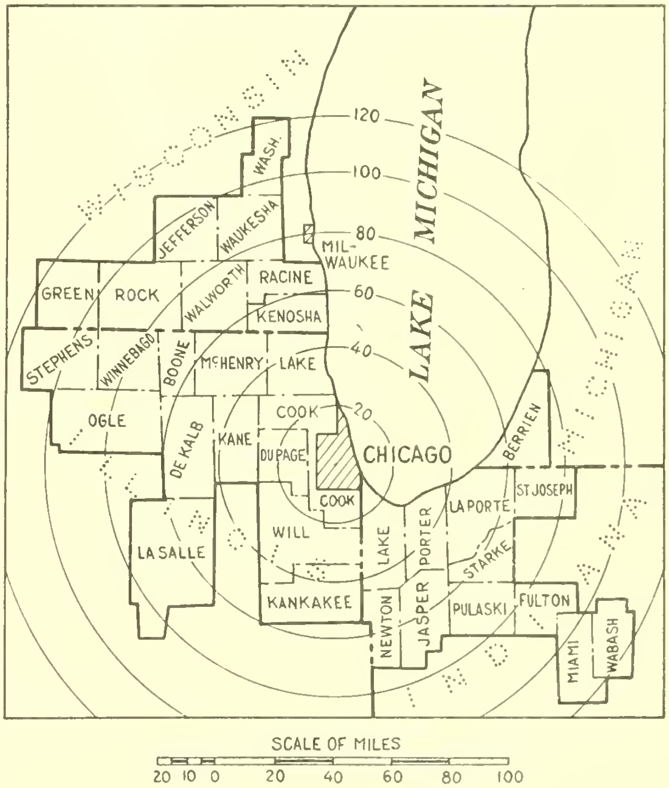


FIG. 1.—The Chicago milk shed

1. The truck passes the producer's gate, thereby furnishing a service which can not be duplicated by the railroads.
2. The driver of the truck acts as the producer's agent from the farm to the city dealer and performs a marketing as well as a transportation service.
3. Shipment by truck instead of by railroad reduces the number of handlings from half a dozen or more to only two.
4. The trucks lose fewer cans than the rail carriers.

5. An estimated saving of 5 cents a hundredweight is effected on all milk trucked in by the elimination of the haul from the railroad milk platform to the city milk dealer.

As practically all of the business formerly handled by the electric roads originated within the short-haul zone, the advantages possessed by the motor truck have enabled it to make such inroads as finally to force the discontinuance of the electric service. The steam roads also have lost a considerable part of their short-haul business, but by developing new territory beyond the normal trucking radius have kept the volume of their shipments up to the 1910 level. The evidence of this change in the character of the railroad business is the considerable increase of milk mileage reported by railroads operating in territories where motor vehicle competition is especially keen.

PRESENT SOURCES OF SUPPLY AND METHODS OF TRANSPORTATION

Chicago's milk supply is produced by 350,000 cows owned on 25,000 dairy farms located in northeastern Illinois, southern Wisconsin, and northwestern Indiana. (See fig. 1.) During periods of extreme shortage some milk is shipped from eastern Iowa and southwestern Michigan.

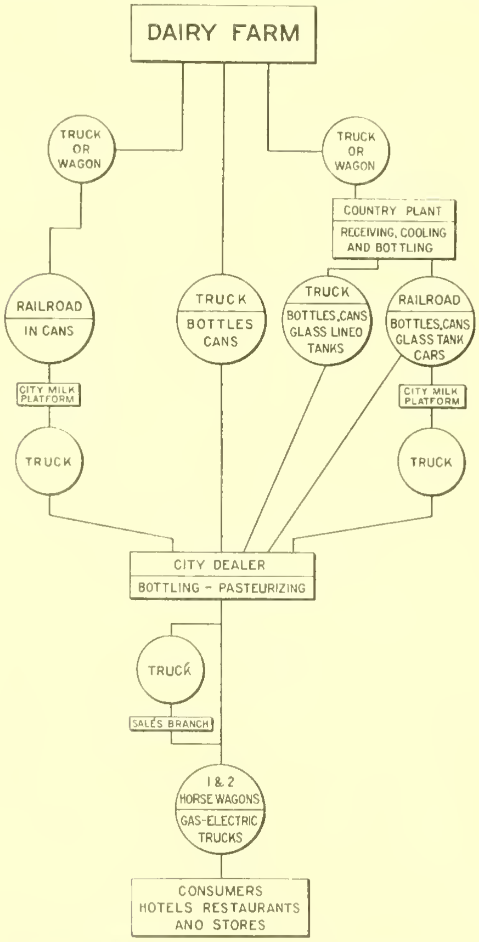


FIG. 2.—Diagram showing stages in the transportation of milk by rail and motor truck to the Chicago market

Approximately 25 per cent of the milk produced is shipped directly either by road or railroad from the producer to the city milk dealer. The larger balance is received from the producers at country milk plants, most of which are operated and owned by the dealers, and which are classified according to their character and purpose as—

1. Country bottling plants, where milk is received, pasteurized, cooled, and bottled for shipment to the city.
2. Receiving stations, where milk is weighed in. cooled, and prepared for shipment in cans or glass-lined tank cars or trucks.
3. Cream stations, which are smaller and less expensive stations, where cream is purchased and pasteurized.

The milk that is collected at these country plants and that which is shipped by railroad must first be hauled to the plant or railroad shipping point. This is called country hauling and is often accomplished individually or by groups of farmers who agree to carry the milk in turn or employ a driver to haul it. Then there is the man who makes a business of country hauling. He usually operates in territories of heaviest production and covers a regularly laid out route, employing for the purpose trucks during most of the year, but substituting horses in times of heavy snowfall. Practically all milk that is shipped by railroad is handled first on a country haul whether it passes through a country plant on its way to the city or not; that which is shipped by truck moves directly to the city except when it is received at a country plant. The 8 or 10 gallon can is the capacity in general use in this area.

The milk which is shipped directly to the city and that which is handled through country plants subsequently finds its way to the Chicago market by one of the following methods:

I. By truck.
    1. Direct from the farm in 8-gallon cans.
    2. From receiving stations in glass-lined tank trucks.
    3. In bottles from certified milk farms.
    4. In 8 or 10 gallon cans from receiving stations.
II. By railroad.
    1. Direct in 8 or 10 gallon cans.
    2. From receiving stations in glass-lined tank cars.
    3. In bottles from country bottling stations.
    4. In 8 or 10 gallon cans from receiving stations.

The several methods of transportation, the number of handlings, and the character of the containers used for each method are shown diagrammatically in Figure 2.

DELIVERY TIME AND REGULARITY OF TRUCK AND RAIL SERVICE

Country bottling plants.—The delivery time under this system from producer to consumer for the oldest portion of each day's supply—namely, the evening milking—is approximately 36 to 44 hours. The general practice is to take the evening's and morning's milk to the country plant between the hours of 6 and 9 a. m. At the plant it is weighed, clarified, pasteurized, cooled, and bottled in the forenoon. In the afternoon of the same day it is loaded into cars, iced, and shipped to the city. The cars are switched at once to the industrial sidings controlled by the dealers, unloaded directly onto their platforms, and transferred to the waiting wagons for morning delivery. The milk is in transit from four to six hours, depending on the length of the haul and the time consumed in making the railroad transfer.

Can shipments.—When the milk is pasteurized and bottled in the city the practice of the producer is to deliver both night's and morning's milk to the railroad shipping point between the hours of 6 and 8 a. m., where it is picked up by a local milk train and transported to the various city milk platforms. The average haul for can milk approximates 48 miles. The greatest distance from which regular milk trains carry milk in cans is 90 miles. Milk on such trains is seldom refrigerated. It is usually in transit from two to four hours, and an additional hour is required for city hauling.

Trucked-in milk.—Trucked-in milk is usually picked up from small milk platforms placed at intervals along the roadside. During the summer months many trucks cover lanes and side roads, taking milk directly from the farm milk houses. In the winter months and when the frost is coming out of the ground the bulk of the milk is hauled by the farmer to the hard-surfaced road where it is picked up by the truck. On a large milk route a 3½-ton truck can pick up a full load (80 to 120 cans) in one hour. After loading, the truck increases its speed for the remainder of the trip to the city plant. Milk is usually picked up between the hours of 6.30 and 8.30 a. m. during the summer months and one-half to an hour later in winter. An inclosed truck of the type required by the Department of Health is shown in Figure 3; and the points from which milk is shipped by truck are shown in Figure 4.

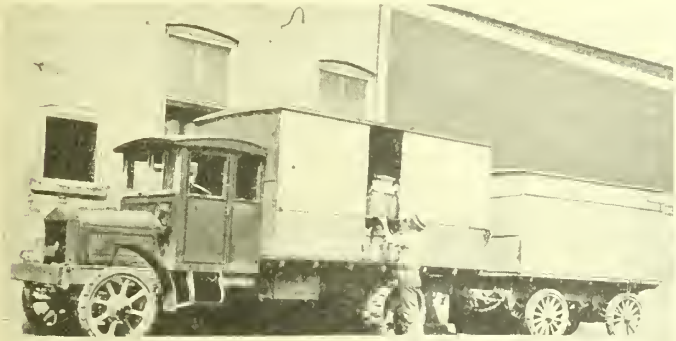


Fig. 3.—Truck with trailer designed to comply with the sanitary regulations of Chicago

Table 2 shows the average trip time of trucks of various capacities engaged in hauling milk in the various mileage zones. The averages which are based on records of 141 trucks show that there is a marked correlation between capacity and speed. Thus it will be noted that the trucks operating from points 20 to 74 miles distant make a higher average speed. The relatively slower speed of trucks operating within the 20-mile zone is undoubtedly due to congestion and the frequency with which grade crossings are met. The latter handicap is especially noticeable on roads entering the city from the south.

Table 2.—*Average trip time in hours for trucks of various capacities hauling in the various mileage zones*

Capacity of truck	Mileage zones						
	5–9	10–14	15–19	20–24	25–29	30–34	35–39
	Trip time						
	Hours	*Hours*	*Hours*	*Hours*	*Hours*	*Hours*	*Hours*
Less than 1 ton	1	--------	--------	--------	--------	--------	--------
1 ton	--------	--------	1¼	--------	1½	--------	--------
1¼ tons	--------	--------	--------	1½	--------	2	--------
1½ tons	--------	--------	--------	1½	--------	3	--------
2 tons	--------	--------	2	--------	2½	3	--------
2½ tons	--------	--------	1¾	2	2¼	2½	3
3 tons	--------	--------	2	--------	3	3	--------
3½ tons	--------	--------	2	2¼	2¾	3	3
5 tons	--------	1½	2	2¼	2½	2¾	3½
Over 5 tons	--------	--------	2½	--------	3	--------	--------

Capacity of truck	Mileage zones					
	40–44	45–49	50–54	55–59	60–64	70–74
	Trip time					
	Hours	*Hours*	*Hours*	*Hours*	*Hours*	*Hours*
Less than 1 ton	--------	--------	--------	--------	--------	--------
1 ton	3¼	--------	--------	3½	--------	4½
1¼ tons	3	--------	--------	--------	--------	4½
1½ tons	--------	3½	--------	--------	--------	--------
2 tons	3½	4	4	--------	--------	--------
2½ tons	3¼	--------	4	4	4¼	--------
3 tons	--------	--------	--------	5	--------	--------
3½	3½	3¾	4	5	--------	--------
5 tons	4	4	4½	5½	--------	--------
Over 5 tons	3½	--------	--------	--------	5	--------

BULK SHIPMENT

Bulk transportation, new as it is, is attracting more attention in Chicago than any other new feature of milk transportation. This method has developed along two lines—the tank truck and the tank car. Each is adapted to certain methods of marketing and each has a distinct sphere of operation.

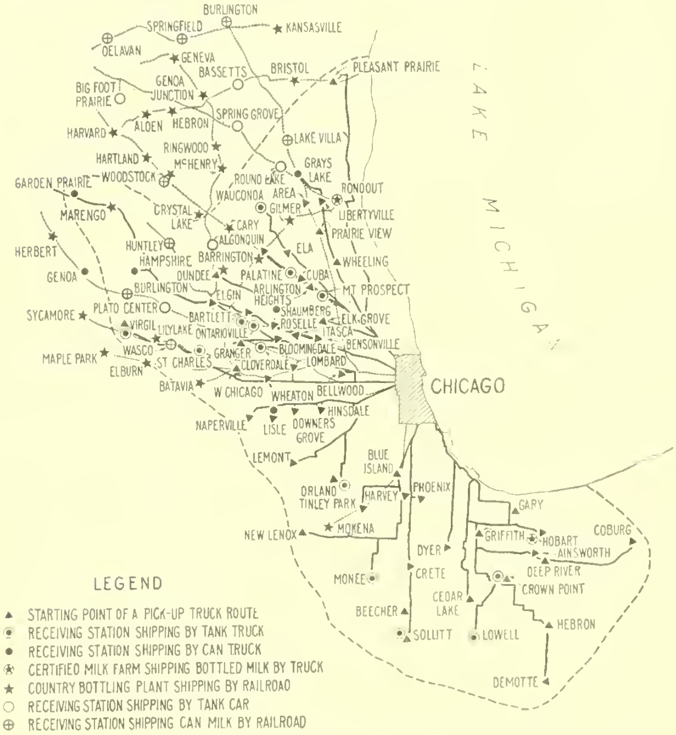


Fig. 4.—Origin and method of transporting all milk originating within motor trucking radius of Chicago

Tank cars.—Tank-car delivery is especially suitable for dairies which are located on industrial sidings and which receive milk from country plants located directly on the railroad. In this case it is possible to load the milk directly from the country station, often by gravity, and to pump it from the car at the city storage plant. If either the dairy or the country station is not provided with rail facilities it is necessary to employ the tank truck as a connecting link, and in such a case it is preferable to use the trucks for direct delivery, unless the distance is beyond the economic range of truck haulage.

Not all dealers, however, can make use of tank-car service, even when they have direct rail facilities, because the quoted rail rate is based on a minimum load of 40,000 pounds, and relatively few country plants can supply this amount daily throughout the year. It is not impossible, of course, to load a tank car at more than one bulk station, but if this is attempted the additional stations may not be handled profitably during the surplus season.

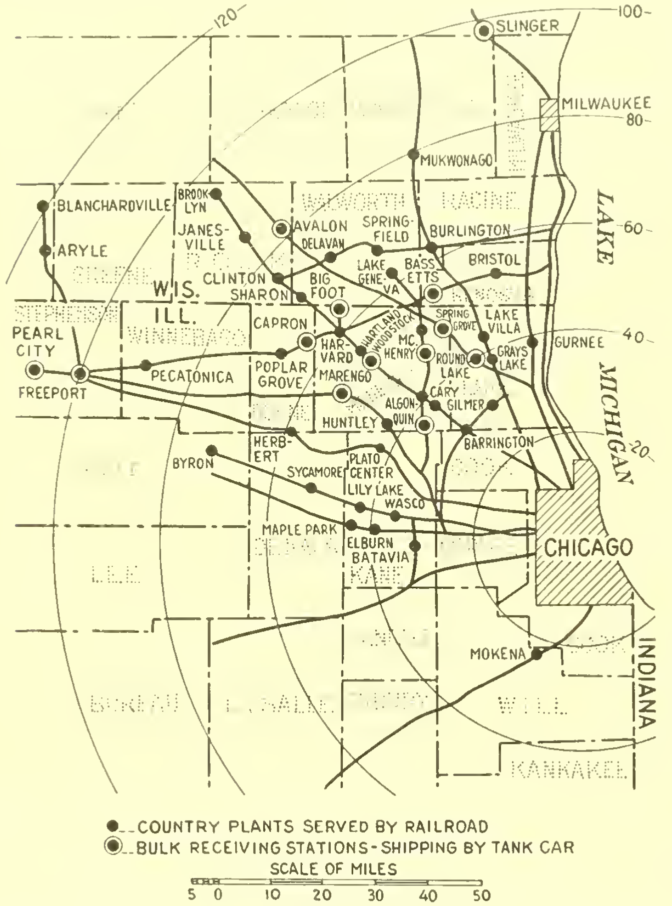


FIG. 5.—Map showing country plants served by railroad in the Chicago dairy district

TABLE 3.—*Railroad tank car installations in the Chicago dairy district*

Stations	Distance	Time en route	Rate per hundredweight	Railroad	Number of cars
	Miles	*Hours*	*Cents*		
Spring Grove, Ill	53.8	3½–4	21	C., M. & St. P.	3
Avalon, Wis	88.9	4 –6	24	do	3
Plato Center, Ill	45.4	2½–3		I. C	2
Slinger, Wis	117.5	6 –7	27	C., M. & St. P.	3
Freeport, Ill	130.0	6 –7	27	do	3
Bassetts, Wis	64.0	4	24	do	4
Big Foot Prairie, Ill	70.0	5 –6		Via electric and C. & N. W.	4
Algonquin, Ill	52.8	3½	21	C. & N. W	4
Capron, Ill	69.5	5 –6	24	do	4
Round Lake, Ill	46.1	3½–4	21	C., M. & St. P.	2
Marengo, Ill.[1]	60.0	4		C. & N. W	3
McHenry, Ill.[1]	50.5	3½–4		do	3
Woodstock, Ill.[1]	51.3	3½–4		do	3
Pearl City, Ill.[1]	123.0	6 –7	27	C. G. W	3

[1] Cars not yet in operation.

The tank car tends to extend the limits of the milk shed by providing relatively cheap transportation from distant points. Several are now being operated, for instance, into Wisconsin districts which have never heretofore shipped market milk to Chicago. The railroads are willing to quote reduced rates on the bulk shipments because the capacity of a tank car, which is approximately 51,000 pounds, is more than twice the capacity of a car loaded with 8 or 10 gallon cans. For example, the rate charged per hundredweight for milk shipped from Slinger, Wis., in tank cars is only 27 cents. (See Fig. 5.) Slinger is 117.5 miles from Chicago, and the rates for milk shipped in cans or bottles are 45 cents per hundredweight for milk in 10-gallon cans, 49 cents for milk in 8-gallon cans, and 72 cents per hundredweight for bottled milk.

There are 14 tank-car installations in the Chicago district all of which are now in operation or shortly will be. The shipping distance for these installations varies from 45 to 130 miles, the shipping time from two and one-half to seven hours and the number of cars employed from two to four, as shown in Table 3.

Tank trucks.—The glass-lined tank truck is admirably suited for the hauling of milk from country stations to city plants, especially when neither is located directly on a railroad and when the distance is not beyond the economic range of truck haulage. Its service is limited to the picking up of a load of milk at a receiving station, although it is possible, as in the case of the tank car, to secure the load from more than one station. It can not be successfully used to collect small quantities of milk along the road, principally because the pooling of the milk of various producers in one tank prevents the proper sampling and checking of sour milk should there be any. Another reason is that the frequent opening of the tank would lead to contamination.

The capacities of the tanks range from 800 to 2,000 gallons, the most practical size being 1,250 gallons. Some of the larger tanks when loaded to capacity exceed the maximum highway weight limits in some States, and it has been found expedient for this reason to mount them on semitrailers. This distributes the load over six wheels and meets all regulatory restrictions. (See Fig. 6.)

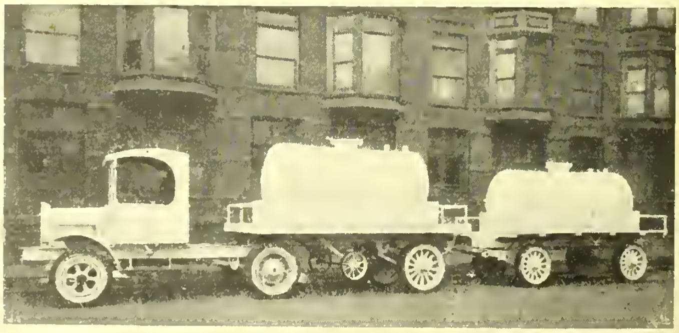


FIG. 6.—The use of tank trucks and trailers has attracted considerable attention in the Chicago area

In the majority of cases tank trucks are owned by the city distributors, although a few motor trucking companies use them on a contract basis. It is not practicable to ascertain the cost of operating the trucks which are owned by the dairies. The rates charged by the hauling contractors are slightly higher than comparable rail rates for tank-car service. From Lowell, Ind., for example, the present tank-truck rate is 23 cents a hundredweight for the distance of 42 miles, as compared with a tank-car rate of 21 cents per hundredweight for distances of 40 to 45 miles. It is apparent, therefore, that a dairy operating its own trucks could easily equal the rail rate.

There are 20 glass-lined units operating over the highways in the Chicago area. A number were put into service during 1924 and there seems to be a growing tendency to convert country bottling plants within 40 or 50 miles of the city into receiving stations and transport the milk in bulk to city dairies and bottling plants. The principal reasons for this change are the high rail rates on bottled milk and the completion of many new highways within trucking radius of the city. Within the past two years no less than a dozen country bottling stations have been converted into bulk receiving stations for these reasons. Those well located within trucking range on good highways are using tank trucks, while the more distant stations are operating tank cars.

A list of the tank truck installations now in operation in the Chicago area is given in Table 4. All of these trucks operate within a radius of from 20 to 60 miles, the average length of route being 34.2 miles. In Tables 5 and 6 are shown the sizes of the tanks used on the trucks of the several capacities and the mileage zones in which the trucks of the various sizes are operated.

TABLE 4.—*Tank truck installations in the Chicago dairy district*

Receiving station	Route	Distance	Operating time to Chicago	Tank capacity	Number	Capacity of truck
		Miles	*Hours*	*Gallons*		
Wauconda, Ill.	Rand Road via Milwaukee Avenue.	46	3¾	1,500 1,250 1,250 1,250	2	5-ton trucks and 2 trailers.
Palatine, Ill.	Northwest Highway and Milwaukee Avenue.	33	2½	1,250 1,250	1	5-ton truck and trailer.
Sollitt, Ill.	Dixie Highway and Halsted Street.	32	2½	1,000	1	3½-ton truck.
Monee, Ill.	Via Chicago Heights and Halsted Street.	32	3	1,500	1	7½-ton tractor and semi-trailer.
Mount Prospect, Ill.	Northwest Highway.	20	2	1,500	1	10-ton tractor and semi-trailer.
Crown Point, Ind.	Jackson Highway via Hammond.	32	2¾	1,500	1	5-ton tractor and semi-trailer.
Lowell, Ind.	do	42	3¾	1,250	1	5-ton truck.
Bartlett, Ill.	Via Lake Street	25	2	1,250	1	Do.
Ontarioville, Ill.	do	22	2	1,250	1	Do.
Virgil, Ill.	St. Charles Road via Roosevelt Road.	50	4	1,250 1,000	1 1	3½-ton truck. 2½-ton truck.
Orland, Ill.	Via Ninety-sixth and One hundred and eleventh Streets and Vincennes Avenue.	29	3	1,500 1,000	1 1	5-ton truck. 3½-ton truck.
St. Charles, Ill.	Geneva and Roosevelt Road.	45	3½	2,000 1,630	1 1	10-ton tractor and semi-trailer. Do.
Cloverdale, Ill.	Lake Street	27	2	1,250	1	5-ton truck.

TABLE 5.—*Number and capacity of trucks and tractors upon which are mounted the various sizes of glass-lined tanks*

Size of tanks (gallons)	Number of trucks and tractors of various capacities				
	2½ tons	3½ tons	5 tons	Over 5 tons	Total
1,000	1	2			3
1,250		1	6		7
1,500			2	3	5
1,630				1	1
2,000				1	1
Total	1	3	8	5	17

TABLE 6.—*Number and capacity of tank trucks operating in the various mileage zones*

Mileage zones	Number of trucks of various capacities				
	2½ tons	3½ tons	5 tons	Over 5 tons	Total
20–29		1	4	1	6
30–39		1	2	1	4
40–49			2	3	5
50–59	1	1			2
Total	1	3	8	5	17

COMPARISON BETWEEN TANK CAR AND TANK TRUCK SERVICE

As previously stated the tank truck and the tank car have each their distinct field of service, the car serving best in the long haul areas especially from plants lying immediately upon the railroad, and the truck in the short-haul zones from plants located on good highways. Where facilities are available and circumstances are such that either form of equipment could be used the truck has an advantage over the tank car because of the smaller investment required for tank-truck service. For a given quantity of milk delivered, less tank capacity is required for truck service than for rail service. This is due to the fact that the trucks can complete the round trip between the country receiving station and the city dairy in a day while the cars normally require more than one day.

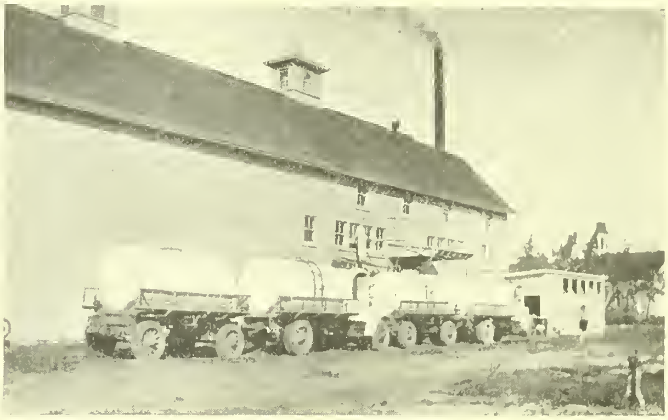


FIG. 7.—The tank trucks are often filled by gravity at the country stations

The tank trucks move out into the country during the early morning hours and are filled at the collecting station, often by gravity, and usually in from 12 to 15 minutes. They leave the station for the city between 10.30 and 11 in the morning, after all milk from the surrounding country has been brought in and cooled, deliver their loads in the early afternoon and are then made ready for the return to the country the following morning.

The use of tank cars, on the other hand, requires that there shall be at least one extra car, generally two and sometimes three, for each car unloaded daily at the city plant. Within a radius of 40 miles two cars often suffice for a single installation. For greater distances, however, there is generally a car loading at the country station and another in transit for each car unloaded at the city end of the haul.

To illustrate the comparative investment required for the two types of equipment, assume that a city dealer receives 6,000 gallons of milk daily which can be shipped either by rail or by truck over hard-surfaced roads. If the milk is hauled in tank trucks, three trucks and two trailers will be required to carry the five 1,250-gallon tanks, and the cost of the outfit will be $24,250. To transport the same quantity by rail will probably require three tank cars of 6,000 gallons each, and the cost will be $45,000. If two tank cars can be made to serve the cost will still exceed that of the tank-truck outfit by 25 per cent.

The tank truck is more flexible for handling the variable output of country stations than the tank car. During the late summer and fall when milk production decreases the dealer operating tank trucks and trailers can suspend the operation of a truck or trailer, during slack periods. In the case of tank-car delivery the rates which apply to these shipments are for a minimum carload of 40,000 pounds. It is a serious matter in times of shortage to operate partly filled cars at the full rate, and it is, therefore, imperative to operate tank cars in connection with stations having a 40,000-pound minimum volume.

MOTOR TRUCK AND RAIL MILEAGE AND RATES

Country hauling.—The rates charged for country hauling are directly proportional to the distance and condition of the roads on the various routes. From Table 7 it will be seen that those dairymen living along unimproved highways pay from 25 to 50 per cent more than those in sections where improved roads have been completed. The value of an improved road to a community can easily be measured in the light of these facts.

TABLE 7.—*Rates paid for country hauling at various stations of a large Chicago dairy*

Station	Types of roads	Distance	Hauling charge per hundredweight
		Miles	*Cents*
Dundee, Ill	Concrete and gravel	3–15	12, 15, 18
Herbert, Ill	Gravel, mostly dirt	3–12	15, 20, 25
Batavia, Ill	Concrete and gravel	3– 9	15, 18, 20
Byron, Ill	Gravel and dirt	3– 9	15, 20, 25
Wauconda, Ill	Concrete and gravel	3– 8	10, 12, 14
Mokena, Ill	Gravel and dirt	5–10	20, 25
Gurnee, Ill	Concrete and gravel	5–11	18, 20
Poplar Grove, Ill	Mostly dirt, some gravel	3– 9	15, 20, 25
Ringwood, Ill	Gravel	5–12	15, 20
Maple Park, Ill	Gravel and dirt	5–10	15, 20, 25
Clinton, Wis	Concrete, gravel, dirt	3–10	12, 15, 20, 25
Brooklyn, Wis	Gravel and dirt	3– 9	15, 18, 25
Janesville, Wis	Concrete, gravel, dirt	3–12	15, 20, 25
Bristol, Wis	Concrete and gravel	3–10	15, 20
Capron, Ill	Mostly gravel	3– 7	8, 10, 12, 15, 17
Big Foot Prairie, Ill	High type of gravel	3– 6	10, 12, 14, 15
Harvard, Ill	do	3– 9	10, 12, 15, 18
Crystal Lake, Ill	Mostly gravel	3–10	15

The rates range from 8 cents per 100 pounds for relatively short distances over improved roads to 25 cents for a 12-mile haul. There are, however, glaring inconsistencies in some of the rates charged for specific distances. Where the rates seemed high and out of line it was found that the trucks operated over roads which had not been improved. For example, the milk delivered to the station at Dundee is transported over gravel and concrete roads for distances of 3 to 15 miles at a rate of from 12 to 18 cents per hundred pounds, while twenty-five miles west of Dundee at Herbert the haulage rate over dirt roads for distances of from 3 to 12 miles is 20 to 25 cents.

THE EFFECT OF TRANSPORTATION RATES ON THE EXTENSION OF THE MILK SHED

Transportation rates and the extension of the milk shed supplying Chicago are rather closely related. The phenomenal development of production to the northwest is largely due to the energetic work of certain railroads running through that section in securing and developing fluid milk shipments. While the railroads have tended to push the dairy district out beyond its present limits the motor truck has intensified production within fifty miles of Chicago and this has had a counter effect.

The cost of shipping 100 pounds of milk in 10-gallon cans to Chicago by railroad is shown in Figure 8. By plotting the railroad rate at different shipping stations on the various railroads and connecting by means of a line the stations having the same rate, the rate zones for rail shipments to Chicago are outlined. The production of milk by counties in thousands of gallons is also shown. As shown by the map there is no uniformity in the system of rates.

The largest portion of Chicago's milk supply is produced within 75 miles of the city, in the territory bounded by the rate zone of 40 cents per hundred pounds. There is sufficient milk produced in the zone bounded by the transportation rates of 40 and 45 cents to meet all needs in the immediate future, and some of this milk is already finding its way to Chicago in glass-lined tank cars.

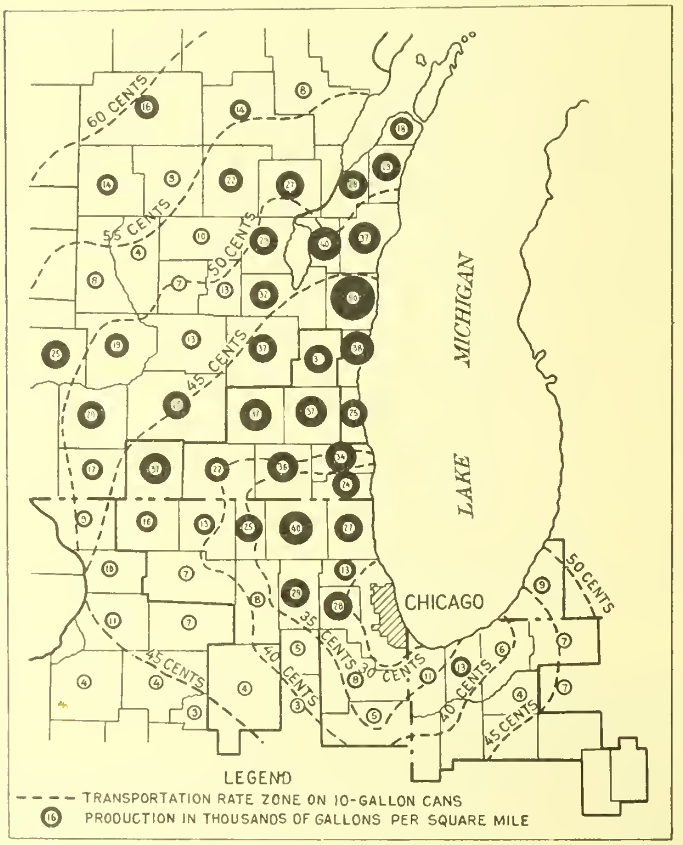


FIG. 8.—Map of the Chicago dairy district and tributary area showing the production of milk by counties in thousands of gallons per square mile. The cost per 100 pounds of shipping milk by rail in 10-gallon cans is indicated by the dotted lines forming the outer limits of the rate zones. Note the heavy production between the 40 and 45 cent rate lines. Most of this milk is now condensed or made into butter or cheese

Truck and rail rates on direct shipments.—Milk transportation rates by truck conform quite closely to the rates charged by the rail carriers operating in the adjoining territory. It must be remembered, however, that on railroad shipments the dealer must pay the cartage from the railroad milk platform to his plant, in addition to the rail tariff. With the truck this cost is absorbed in the rate charged.

Truck rates are more nearly proportional to distance than rail rates for the first 30 miles. The general tendency among the rail carriers is to charge a blanket rate for the first 25 or 30 miles and, for greater distances, a proportional increase according to distance. These characteristics of the truck and rail rates are illustrated by Figure 9 and the comparative rate schedule in Table 8.

TABLE 8.—*Truck and rail rates on direct shipments*

Railroad			Truck		
Distance	Rate on cans of not exceeding 10 gallons capacity	Rate on cans of not exceeding 8 gallons capacity	Distance	Range of rate per 8-gallon can [1]	Average rate per 8-gallon can
Miles	*Cents*	*Cents*	*Miles*	*Cents*	*Cents*
1-25	30	27	1-10	15	15
26-30	32	29	11-15	16-18	17
31-35	33	30	16-20	18-20	19
36-40	35	32	21-25	20-24	22
41-45	36	33	26-30	22-28	25
			31-35	24-30	27
			36-40	26-34	30
			41-45	28-36	32
			46-50	30-40	35

[1] Practically all cans on trucks in Chicago dairy district are of 8-gallon capacity.

While there seem to be inconsistencies in some of the truck rates for the same mileage zone, a tabulation of the rail rates would show even less uniformity. A close investigation of the truck rates reveals a marked increase in the rate where the truck route follows poor roads.

In the various sections of the dairy district there are as many different systems of computing rates for hauling milk as there are motor trucks. Certain operators charge by the gallon, others by the hundredweight, while the most common method is a stated rate per 8-gallon can. Still another method used by hauling contractors handling bulk shipments in tank trucks is to charge a stipulated amount for all milk up to 2,000 gallons. One dealer has a contract with a trucking company operating two tank trucks to haul his milk at the rate of 30 cents per hundredweight for a 50-mile haul. The railroad rate for this country plant is 33 cents per hundredweight, or 3 cents more than the truck rate, and, in addition, the city dealer saves the cost of cans and the cost of carting the milk from the railroad milk platform to the city plant. The trucking company owns the trucks and glass-lined tanks and assumes all road responsibility.

Basis for truck rates.—In determining the rate, the length of the haul naturally is one of the prime considerations; road conditions constitute another. The fact that milk in cans is very perishable and bulky must also be taken into consideration, and also the fact that where a complete delivery is made from the dairy farm or roadside platform to the city dealer a service is rendered which is not duplicated by the railroad. This additional service should be considered in fixing the rate. Practically no rates have been established on the basis of cost, plus a reasonable profit. A satisfactory rate must be one which is low enough to attract business and high enough to offer a reasonable profit. Where conditions do not permit the establishment of such a rate, a route should not be established.

THE VALUE OF HARD SURFACED ROADS

Good roads are a prerequisite to successful motor truck operation. The unfortunate operator who tries to maintain his service over highways which are virtually impassable for a part of the year finds that his daily operating costs far exceed the average normal expenses, and eventually he realizes that his profits are seriously affected. Very few operators appreciate the increased expense which results in this way from bad roads; but that there are some that do consider the condition of the highway in proportioning haulage charges is illustrated by the rates given in Table 9, which also prove the value of hard-surfaced roads in keeping down transportation rates.

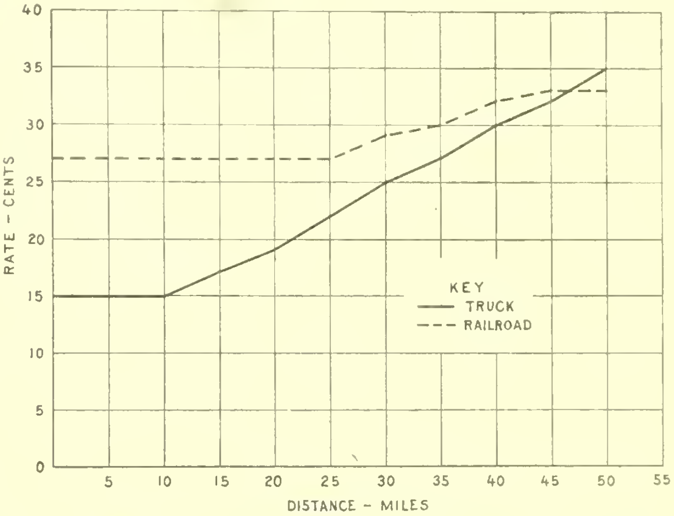


FIG. 9.—Diagram showing the average rates for the hauling of milk various distances into Chicago by motor truck and railroad

TABLE 9.—*Road types and their effect on haulage rates*

Station	Type of hauling	Type of road	Distance	Hauling charge per hundredweight
			Miles	*Cents*
Garden Prairie, Ill.	From station to Chicago in cans.	Concrete	66	35
Herbert, Ill	Country hauling	Dirt and gravel	3-12	15-20-25
Sollitt, Ill	From station to Chicago in tank truck.	Concrete	32	[1] 17. 15
Lake Geneva, Wis.	Country hauling	Concrete and gravel.	5-15	15-20
Lemont, Ill	"Pick-up" route	Dirt and concrete.	19	36. 4

[1] 1½ cents a gallon.

As an instance of the value of hard roads, a large Chicago dairy, which had a receiving station on a rail line which was no longer operated, believed for a time it would have to abandon its plant. As the section in which the plant was located is a highly productive dairy district, the company made every effort to find another means of transportation. About this time the Rand road was completed, furnishing 46 miles of hard-surfaced road to the city. An installation of two tank trucks and two trailers is now operated daily between this station and the Chicago plant with the result that the highway has saved the closing down of the plant and the community retains a profitable place to market its milk.

As another instance, a large Chicago dairy operated glass-lined tank trucks between Chicago and their country plant at Bassetts, Wis., a distance of 77 miles. Of this distance approximately 68 miles are hard surfaced and the remainder unimproved and extremely difficult to traverse during prolonged wet spells. The operation was very successful in good weather, but the installation finally had to be abandoned because of the miring of trucks in mud holes along the unsurfaced portion of the route in wet weather, and the milk is now hauled from this plant in railroad tank cars. The officials of the company state that the operation of tank trucks was successful in every detail except over the 8 or 9 miles of unimproved road. The regularity of service so essential in milk transportation could not be maintained under these adverse conditions.

Many highly productive dairy districts have been developed in communities which have inadequate rail service, by the building of hard roads which enable the motor truck to collect daily milk and other farm produce for the near-by city.

ZONES OF OPERATION AND KINDS OF TRUCKS USED

At the close of 1924 there were 141 motor trucks engaged in the transportation of milk into the city of Chicago. While these trucks are handling considerable milk from bulk receiving stations the largest volume comes direct from the producer. Much of this milk formerly moved by rail, uniced, in regular baggage cars from a zone the maximum radius of which was 75 miles. It is estimated that there are about 1,800 railway can shippers left and it is highly probable that the number will continue to decrease as the use of the motor truck increases.

Table 10 gives the amount of milk transported in cans, tank trucks, and in bottles and also the number of trucks engaged in each operation. It is highly significant to note that 17 tank units haul approximately 23 per cent of the trucked-in milk.

Table 10.—*Methods of trucking milk and the daily amounts transported*

Means of transportation	Units	Number of units	Number of gallons	Number of trucks	Percentage of total trucked-in milk	Average number of gallons per truck
"Pick-up" can truck.	8-gallon cans	10, 682	85, 456	122	76	700. 4
Tank trucks and trailers.	Various capacities 1,000–2,000 gallons.	20	26, 630	17	23	1, 566. 4
Certified bottled milk.	Cases of 12 quarts each.	209	627	2	1	313. 5
Total			112, 713	141	100. 0	860

Table 11.—*Number of milk trucks operating in the various mileage zones*

Zone	Number of trucks	Percentage of total number
Miles		*Per cent*
0–9	1	0. 7
10–19	7	4. 9
20–29	47	33. 3
30–39	39	27. 6
40–49	26	18. 4
50–59	17	12
60–69	2	1. 4
70–79	2	1. 4
Total	141	100

The distances over which these trucks operated ranged from 8 to 75 miles. As shown in Table 11, only eight trucks or approximately 5 per cent come from points less than 20 miles away. So much land has been subdivided and taken over by golf links that very little milk is produced within the 20-mile zone. The land given over to agriculture within this zone is devoted to truck gardening which is more intensive and profitable than dairying on such high-priced land. The largest number of trucks operate in the 20-29 mile zone while only four trucks or less than 3 per cent operate over 60 miles.

The motor units engaged in milk hauling are of varying capacities, ranging from ¾-ton trucks to 10-ton tractors. The 2½-ton and 3½-ton sizes are by far the most popular capacities as shown by Table 12. They are well adapted to the average route and can make excellent speed over hard roads after their load has been picked up.

Table 12.—*Number of milk trucks, by capacities—Chicago dairy district*

Capacity	Number of trucks	Percentage of total number
Tons		*Per cent*
Less than 1	1	0. 7
1	6	4. 2
1¼	4	2. 8
1½	4	2. 8
2	9	6. 4
2½	43	30. 5
3	5	3. 6
3½	42	29. 8
5	22	15. 6
Over 5	5	3. 6
Total	141	100. 0

The following table gives the carrying capacity in cans of the various sizes of trucks:

Table 13.—*Average number of 8-gallon cans loaded on trucks of various capacities*

Capacity	Number of 8-gallon cans	Capacity	Number of 8-gallon cans
Tons		*Tons*	
Less than 1	20–25	3	70–90
1	30	3½	80–120
1¼	35	5	100–140
1½	40	Over 5	150–200
2	50	Tractor and semi-trailer	150–200
2½	60–70		

RETURN LOAD BUSINESS NOT DEVELOPED

Very few milk truck operators carry any return load. Occasionally a load of feed or farm implements is carried, but this type of business has never grown to the proportions it has in a number of other cities, especially Baltimore. One reason is that the city is so large that it takes too much time to gather up a load. Another is that the success of a well-organized return-load movement depends largely on the establishment of a central freight depot, and as yet there has been no attempt to establish such a depot.

MILK TRUCK REGULATION

The Department of Health of the City of Chicago has recently made a thorough investigation with reference to transportation of milk into the city by motor truck.

The methods and equipment used in picking up milk by truck, in some instances, were quite unsatisfactory to the Health Department and in view of this fact it has been found necessary to enforce the following regulations:

1. All milk trucks used in the transportation of milk or cream must be completely inclosed, having permanent tops and sides constructed either of wood or metal.
2. A list giving the names and post office addresses of all dairymen whose milk is hauled regularly and names of dealers supplied must be submitted to the department at least every 60 days.
3. A time schedule is to be arranged similar to that used by rail transportation companies, so that each farm will be reached at approximately the same time each day, thus obviating the necessity of milk standing out unprotected for any considerable length of time before the arrival of the truck.
4. Arrangements must be made to load all milk from raised covered platforms, whether it be on the farm or at certain other points where more than one dairyman loads his milk.
5. Trucks must be maintained in a clean and sanitary condition and be cleaned thoroughly after each day's use.

While it is doubtful if milk shipped by truck is received in any better condition than rail shipments, yet marked improvement in quality has taken place since the establishment of loading platforms along the highways. The truck has reduced the number of handlings to a minimum which lessens the chance of contamination greatly.

There is no common carrier regulation of motor trucks of any kind at present in force in the State of Illinois, and the only license fees paid are the ordinary State motor truck license and the wheel tax. The latter is only paid where the truck is owned in some incorporated city or village which levies such a tax.

OVERLOADING

There is a very small percentage of overloading in the milk hauling business. Most of the trucks are purchased with the view of taking care of the surplus milk produced in May and June, and they are, therefore, of sufficient capacity to carry their normal loads and come within all regulations. Some of the larger tank trucks were found to be overloaded but in all cases this has been remedied by mounting the tanks of large capacity (1,500 gallons or more) on semitrailers thereby distributing the load over six wheels.

Milk truck operators from northwestern Indiana are demanding truck legislation which will give them an idea of what lawful weights can be carried in the spring of the year when moderately loaded trucks are ordered from the highways. In order to operate at such times most Indiana operators find that they must own or hire surplus trucks of 1-ton apacity, which from the operator's point of view is a very unsatisfactory condition. The real solution to the problem is more hard-surfaced roads which will allow trucks of the larger capacities to operate freely at all seasons of the year.

FIELD METHODS USED IN SUBGRADE SURVEYS

(Continued from p. 101)

RECOMMENDATIONS BASED ON THE SURVEY

In general the shrinkage of the soil even in sections included in the 9 miles referred to above was found to be of no serious consequence. It was decided, therefore, that the greatest attention should be devoted to securing adequate drainage.

Before making recommendations as to the design of the pavement, the adjacent sections of paving in good condition were studied. Assuming soil conditions, traffic and climate to be the same on a pavement project in the same region, it is probable that the same general type would be adequate where similar conditions existed in the proposed project.

The immediate subsoils of the Olympic, Aiken, Salkum, Winlock, Kelso, and Felida series were compact. This impervious layer restricted drainage and made it necessary to recommend the laying of a grade line which would place the excavation beneath it. Otherwise deep side ditches should be provided. This was apparent especially in the Kelso formation in the vicinity of Ostrander. The impervious subsoil was from 2 to 8 feet below the surface, and beneath it was found a friable silt and fine sand stratified with some clay. This lower subsoil drains well and makes a hard, compact, nonshrinking subgrade.

From Woodland to Carrolls the road was built through sand, solid rock, loose rock, etc. With the exception of some sliding ground, these materials, it was decided, should afford an excellent foundation for a pavement.

The Puget series vibrates under train traffic. This was especially noticeable at Ostrander where fills of this series have settled considerably and caused serious cracking and deformation of the reinforced concrete overhead crossing over a logging railroad. The quaking, however, was not considered sufficient to reduce materially the life of the proposed pavement provided the subgrade was permitted to reach ultimate settlement.

The liberal use of culverts and tile drains and the construction of deep side ditches was considered to be the best means of preventing failures from inadequate drainage. Additional subbase seemed necessary only where the existing crushed stone surfacing had been removed or was too thin. This surfacing was placed by the State, in line with its stage-construction policy, when the grade was built, and in most places it was preserved intact to serve as a subbase for the new pavement.

Where the lineal shrinkage of the soil exceeded 5 per cent it was recommended that protective measures be adopted; and the three methods recommended were the use of the thickened-edge type of pavement, the building of a 4-inch subbase or the use of steel reinforcement.

The State adopted a combination of all three methods. The pavement which was completed in the fall of 1923 is 20 feet wide with a center thickness of 6½ inches and an edge thickness of 9 inches on the entire project with the exception of the 15-mile section between La Center and Kalama where the thin edge, thick center (6″–7½″–6″) design was used. Wherever the soil was found to have a lineal shrinkage factor in excess of 5 per cent the pavement was reinforced, except where the existing crushed stone surface was thought to be adequate to serve as a subbase.

A casual inspection of the pavement a year later, in the fall of 1924, showed it to be in excellent condition.

REPORT FORM

The report form used on the Pacific Highway subgrade survey follows:

1. Brief description of highway.
2. Information in regard to adjacent roads.
    - (*a*) Location.
    - (*b*) Type.
    - (*c*) Subbase and character of soil.
    - (*d*) Year built.
    - (*e*) Present condition.
    - (*f*) Summary.
3. Description of this project: Type, etc.
4. Topography.
5. Geological structure.
6. Soil types encountered.
    - (*a*) Data on borings.
    - (*b*) Results of soil analysis.
7. Drainage conditions.
8. Climatological data.
9. Traffic census.
10. Recommendations.
    - (*a*) Drainage—surface and subsurface.
    - (*b*) Pavement design—reinforcement or subbase.
11. Remarks.
12. Photographs.

CHANGE IN SOIL TESTING METHODS CONTEMPLATED

In making a subgrade survey it is essential that the various soil types occurring along the proposed locations be determined, as well as the physical properties of the respective types. The determination of these two factors requires that a large number of soil samples be taken and analyzed in the laboratory.

It would seem advisable to adopt any means by which the time and labor involved in sampling and analyzing could be re uced to a minimum, and, as pointed out elsewhere in this issue, the surveys of the United States Bureau of Soils which cover a large part of the United States would seem to be of inestimable value to the highway engineer for this purpose.

Unfortunately the test methods used by the Bureau of Soils, especially in the mechanical analysis test, differ from those which have been standardized in highway engineering laboratories. The former involve the use of millimeter sieves for separating the fractions of the soil, while the latter are based on sieves the meshes of which are measured in English units. On account of this difference in practice materials which are classified as clay, silt, loam, and sand by the Bureau of Soils are not identical with the materials to which the same names are applied by highway investigators, and this difference stands in the way of a convenient use of the large amount of useful information acquired by the agriculturists.

The article published in this issue makes out a convincing case for the use of the Bureau of Soils bulletins. It does not seem that a difference in method so slight and so easy to eliminate should be permitted indefinitely to embarass the utilization of such valuable and voluminous data; and the Bureau of Public Roads is contemplating a change in its methods of making the mechanical analysis which will bring them into conformity with the methods of the other Federal bureau. There are a number of considerations both for and against the change which will make it necessary to give the matter very careful consideration, and final action will not be taken until the advisability of the change has been fully demonstrated. In the meantime the bureau will welcome the advice and suggestions of highway and testing engineers either for or against the proposal.

A TEST FOR DETERMINING THE AMOUNT OF SHALE IN SAND USING A LEAD ACETATE SOLUTION

By P. M. Hegdal, Engineer of Tests, North Dakota State Highway Commission

During the summer of 1924 the writer developed a test for determining the amount of shale in sand by the use of a lead acetate solution. The advantage of the method, which is described below, lies in the fact that a very heavy solution is obtained which is not sluggish or sirupy. This is a very desirable quality as the thick liquid, if sluggish, prevents all fine particles of the sand from subsiding regardless of their specific gravity.

The description of the lead acetate method follows:

To about 2 kilograms of ordinary commercial acetate (sugar of lead) add about 600 or 700 grams of distilled water. Boil in a glazed pan into the top of which a standard testing sieve will fit, until it is all dissolved. This solution must be kept over a flame or hot plate while the testing is being done as at ordinary temperatures it is a solid. It should be kept near the boiling point and its specific gravity should be kept about 2.40 (approximately) by boiling off water to increase, and by adding distilled water to decrease its weight.

Use that portion of a 500-gram sample of washed dried sand which is retained on a No. 50 sieve.

Sprinkle the sand to be tested into the hot solution, stirring in the floating sand. All the shale will float and can best be skimmed off with a spoon fashioned from a strip of 20-mesh brass screen 2 by 6 inches in size, which may be emptied by dipping it into a pan of hot water. The shale should then be washed in warm water till clearly dried and weighed, and the weight in grams divided by five gives the percentage of shale.

To recover the lead acetate, set a No. 50 sieve in the top of a second pan of the size mentioned above and dump the hot solution into the sieve. The sand will be retained on the sieve and can be removed from it by washing with warm water.

If crystalline C. P. lead acetate is used, heat it without the addition of water to prepare the solution.

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ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583. Report on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916. 10c.
*660. Highway Cost Keeping. 10c.
*670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691. Typical Specifications for Bituminous Road Materials. 10c.
*704. Typical Specifications for Nonbituminous Road Materials. 5c.
*724. Drainage Methods and Foundations for County Roads. 20c.
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
No. 1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1259. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULAR

No. 94. TNT as a Blasting Explosive.

FARMERS' BULLETINS

No. *338. Macadam Roads. 5c.
*505. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *727. Design of Public Roads. 5c.
*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JUNE 30, 1925

STATES	FISCAL YEARS 1917-1924			FISCAL YEAR 1925									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1924			PROJECTS COMPLETED SINCE JUNE 30, 1924			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 4,598,721.63	$ 2,186,247.54	464.1	$ 1,371,376.08	$ 675,950.32	147.7	$ 15,450,912.60	$ 7,494,856.47	804.8	$ 300,107.34	$ 146,749.34	24.8	$ 2,290,029.33	Alabama
Arizona	8,338,365.41	4,287,683.88	527.8	1,241,758.02	728,436.06	86.0	2,186,015.76	1,346,463.82	158.1	279,009.17	170,502.49	32.9	2,018,785.75	Arizona
Arkansas	11,094,751.31	4,424,345.53	944.4	2,215,438.77	955,836.10	104.5	7,175,728.71	3,252,163.54	387.4	625,385.62	279,793.16	43.4	1,414,425.57	Arkansas
California	12,999,075.03	5,647,148.17	533.7	9,347,100.96	5,072,101.44	361.1	9,322,361.01	4,412,288.58	230.4	1,113,234.62	633,572.38	36.3	3,800,831.43	California
Colorado	8,108,070.31	4,029,898.97	502.6	3,768,633.63	2,037,915.37	148.6	3,555,355.79	1,844,058.78	158.9	1,227,702.32	543,845.34	53.4	2,477,399.54	Colorado
Connecticut	3,062,872.02	1,269,558.60	73.6	1,495,767.27	549,810.06	28.0	1,993,218.91	606,847.75	30.9	309,513.05	98,151.61	6.2	1,331,627.98	Connecticut
Delaware	3,056,832.22	1,007,714.83	72.6	1,224,727.59	488,475.82	34.6	489,503.74	213,581.10	12.3	882,478.16	332,834.76	18.9	62,548.50	Delaware
Florida	961,134.07	461,470.92	48.8	1,998,139.65	944,017.06	47.5	8,787,223.95	4,279,162.51	251.8	116,547.09	91,547.09	0.1	1,403,567.43	Florida
Georgia	17,167,373.32	7,955,805.20	1214.2	2,988,629.05	1,450,561.26	264.1	11,245,752.26	6,544,980.07	740.9	523,497.72	228,475.30	34.6	1,253,154.17	Georgia
Idaho	8,161,697.92	4,092,395.52	506.8	1,212,978.88	722,936.74	93.3	2,454,035.02	1,535,561.67	154.9	414,504.97	259,857.84	37.1	1,003,887.23	Idaho
Illinois	26,964,706.06	12,279,546.33	804.7	13,045,775.04	6,360,529.95	431.5	7,979,727.96	3,863,458.50	270.4	487,835.78	243,917.89	17.4	3,880,518.33	Illinois
Indiana	7,677,444.16	3,655,540.97	225.7	6,061,728.49	2,906,914.71	196.4	14,852,730.92	7,195,558.57	432.9	848,584.16	418,630.58	23.2	2,074,440.17	Indiana
Iowa	23,195,778.19	9,237,031.86	1582.9	4,075,507.02	1,870,461.13	314.0	7,506,888.60	3,315,441.22	434.6	939,082.01	457,695.00	89.5	2,525,903.79	Iowa
Kansas	17,084,136.48	6,043,176.80	502.7	9,315,559.29	3,712,096.52	328.7	12,808,547.40	6,388,167.91	600.3	416,777.30	125,651.21	12.8	2,104,656.56	Kansas
Kentucky	10,822,980.31	4,613,947.28	429.4	4,009,343.97	1,592,047.31	155.6	8,298,279.66	3,724,887.09	299.0	590,422.33	295,211.15	40.0	1,557,253.17	Kentucky
Louisiana	8,488,463.18	3,636,143.36	661.2	3,450,961.79	1,643,727.50	266.4	3,571,757.27	1,724,822.29	170.0	370,484.38	185,242.18	2.8	1,072,768.67	Louisiana
Maine	6,911,058.78	3,299,935.38	230.7	1,263,222.53	607,934.95	50.7	1,151,207.40	544,027.42	42.1				1,323,214.25	Maine
Maryland	6,760,044.42	3,213,321.78	243.2	1,372,462.48	636,061.37	51.8	1,932,936.76	815,245.52	57.1	1,323,161.77	589,541.16	85.0	30,563.18	Maryland
Massachusetts	10,191,202.02	4,105,727.22	232.8	3,856,454.20	1,361,934.06	67.8	5,501,324.46	1,424,297.34	84.2	599,497.74	188,790.00	10.9	1,929,149.38	Massachusetts
Michigan	13,434,135.07	6,060,612.23	494.5	2,739,865.73	1,267,704.68	118.1	14,787,520.89	6,990,273.05	475.2	182,013.44	85,902.19	0.3	3,700,505.85	Michigan
Minnesota	24,037,561.24	9,885,843.07	2292.0	6,378,124.65	2,852,798.97	429.2	6,706,509.32	3,182,300.00	683.9	2,799,814.74	882,900.00	146.4	638,727.96	Minnesota
Mississippi	7,888,193.89	3,828,941.39	655.0	2,404,091.90	1,159,761.34	148.4	8,854,700.33	4,421,094.52	491.3	794,462.49	396,184.64	41.6	1,017,251.11	Mississippi
Missouri	11,352,027.70	5,245,899.18	803.6	6,016,128.87	2,973,512.25	315.4	27,236,675.44	11,406,669.15	869.2	651,118.18	247,931.89	18.4	483,902.53	Missouri
Montana	8,867,279.16	4,384,335.12	791.4	1,289,321.25	933,188.03	130.2	2,112,322.39	1,650,333.81	182.8	988,759.09	551,613.73	117.1	5,095,419.31	Montana
Nebraska	7,876,337.16	3,714,691.59	1440.4	1,430,037.20	674,831.91	130.2	8,523,160.76	4,126,635.19	929.3	1,233,565.07	616,782.62	43.0	3,839,973.79	Nebraska
Nevada	3,460,245.52	1,853,624.98	225.6	1,457,220.17	1,234,674.80	131.7	6,116,444.07	4,329,875.30	449.9	80,728.23	70,814.79	16.9	349,407.13	Nevada
New Hampshire	3,076,750.19	1,487,867.68	171.3	1,088,937.67	498,359.29	36.8	882,156.80	426,087.09	28.9	237,118.87	99,631.96	6.2	288,643.08	New Hampshire
New Jersey	7,623,796.12	2,661,531.49	148.7	4,337,562.33	1,159,148.50	70.4	9,090,363.22	3,246,713.23	63.8	1,453,881.96	338,176.67	19.0	118,759.11	New Jersey
New Mexico	5,306,286.46	2,758,849.68	714.3	3,411,712.73	2,155,220.93	367.0	4,353,981.28	2,887,782.68	394.6	205,006.08	126,078.73	12.1	1,846,565.98	New Mexico
New York	18,862,742.49	8,257,844.44	572.7	9,735,027.18	3,971,232.09	258.8	30,005,625.87	10,225,242.60	621.0	6,715,000.00	1,888,481.46	125.0	6,022,443.42	New York
North Carolina	12,567,732.97	5,676,767.66	884.7	8,446,717.44	3,069,696.93	235.1	8,280,397.44	3,553,613.80	217.1	1,332,882.10	638,502.26	43.6	1,054,848.35	North Carolina
North Dakota	9,088,973.11	4,418,505.42	1587.9	1,740,290.71	850,425.05	329.6	3,314,335.51	1,672,819.67	467.1	777,838.74	395,708.64	98.3	2,206,896.22	North Dakota
Ohio	33,122,751.43	11,879,917.99	962.5	8,449,501.38	3,365,075.94	228.6	9,494,395.75	3,397,125.05	285.1	3,306,077.79	1,321,667.35	100.0	2,965,955.67	Ohio
Oklahoma	12,986,865.26	6,888,852.03	497.3	7,800,159.68	3,784,038.31	354.9	7,030,056.22	3,324,602.25	311.9	863,610.54	429,928.50	46.7	864,386.91	Oklahoma
Oregon	12,082,873.17	5,819,093.79	655.6	2,305,315.53	1,323,270.84	139.0	3,016,966.63	1,745,532.24	136.9	610,268.96	366,846.04	38.9	431,084.09	Oregon
Pennsylvania	36,826,248.98	14,114,694.79	729.7	6,229,586.21	2,107,329.18	120.6	30,880,520.14	8,168,048.18	529.2	7,073,904.07	2,045,175.88	155.2	1,526,490.97	Pennsylvania
Rhode Island	1,774,397.25	779,227.96	46.0	854,098.95	340,460.13	16.8	1,798,608.99	518,589.59	26.3	661,890.35	135,450.00	9.0	524,938.32	Rhode Island
South Carolina	9,016,476.73	4,124,045.22	924.4	2,146,871.11	997,222.32	311.5	6,546,967.82	2,938,981.67	373.3	870,524.91	354,687.36	35.1	325,158.44	South Carolina
South Dakota	8,674,597.86	4,244,636.27	989.8	3,416,836.81	1,745,242.73	458.1	7,593,468.47	3,742,326.64	1011.4	354,871.00	180,198.22	54.6	21,296.14	South Dakota
Tennessee	6,805,683.36	3,312,936.07	259.6	6,963,457.63	3,418,143.70	238.3	12,364,278.08	5,687,754.44	422.7				1,327,787.79	Tennessee
Texas	42,341,998.56	16,190,624.91	3122.8	11,778,972.27	4,867,315.21	784.3	25,177,136.37	10,515,352.63	1509.1	2,640,405.17	955,298.80	185.3	3,611,336.45	Texas
Utah	3,304,423.76	1,895,805.92	219.0	2,954,735.66	1,923,030.99	204.1	2,953,903.46	2,014,706.49	226.1	252,461.79	186,821.71	25.3	942,674.89	Utah
Vermont	1,922,114.16	942,769.12	74.4	1,093,060.35	510,125.33	33.4	1,207,289.94	578,117.56	26.6	439,151.67	151,623.81	8.9	716,968.08	Vermont
Virginia	10,035,301.48	4,801,782.43	562.5	3,064,418.53	1,470,215.77	113.7	10,127,047.08	4,560,134.02	348.7	1,705,250.71	760,025.29	65.4	450,508.49	Virginia
Washington	11,384,615.67	5,290,895.45	457.0	1,967,888.51	826,316.42	69.7	3,015,386.61	1,426,900.00	127.3	833,280.25	361,000.00	22.3	1,100,553.13	Washington
West Virginia	6,489,747.95	2,365,041.53	255.6	1,853,452.91	855,261.80	71.1	5,194,890.10	2,178,214.28	149.2	2,066,068.61	754,510.03	49.9	388,409.36	West Virginia
Wisconsin	16,753,903.15	7,441,033.67	1325.3	3,053,237.76	1,478,607.05	126.4	3,570,401.52	1,746,214.36	180.4	1,772,286.08	877,878.00	90.3	4,008,026.02	Wisconsin
Wyoming	6,127,625.61	3,078,098.70	687.6	2,682,193.72	1,660,997.97	294.4	3,766,266.56	2,363,221.79	267.2	594,953.17	382,502.00	54.2	137,477.54	Wyoming
Hawaii							342,277.22	97,440.00	6.5				633,810.00	Hawaii
TOTALS	549,655,391.27	237,852,399.82	32452.9	190,485,399.56	87,801,946.18	9445.4	389,606,481.37	171,448,571.53	17123.0	52,925,019.39	20,892,230.91	2195.3	80,254,851.56	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 105,123,086.96 Federal aid $ 47,606,101.14 Miles 4,587.2

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 6, NO. 6 ▽ AUGUST, 1925

EXPERIMENTAL SECTION OF THE COLUMBIA PIKE SHOWING TYPICAL CRACKS IN A REINFORCED CONCRETE SURFACE

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1925

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THE INTERRELATION OF LONGITUDINAL STEEL AND TRANSVERSE CRACKS IN CONCRETE ROADS

Reported by A. T. GOLDBECK, Chief, Division of Tests, U. S. Bureau of Public Roads

IT is evident from a study of standard designs for concrete pavements that there is a great difference of opinion among engineers as to the proper amount and placement of longitudinal steel. A similar lack of agreement appears in the practice with regard to the spacing of joints; and there is considerable evidence of a lack of appreciation of the essential interrelation between the quantity of steel and the spacing of joints.

That there is such a relation has been definitely shown by investigations made by the Bureau of Public Roads, and it would seem that the facts developed are now sufficiently substantiated to serve as the basis for a rational method of design.

As a preface to the discussion of the method which it is intended to propose it is necessary to have a clear understanding of the various forces which act upon the pavement and a knowledge of the physical properties of the steel and concrete which determine their behavior when they are combined.

It will be assumed therefore that a uniform subgrade has been prepared. The concrete has been placed and begins to harden. Immediately the moisture begins to dry out of it and as it dries it shrinks. The surface dries and shrinks more rapidly than the immediately underlying concrete, thus subjecting the upper skin to tensile stresses which will quickly exceed the slight tensile strength of the material and cause numerous fine surface cracks to form unless the process of shrinkage is halted by the timely beginning of curing.

The curing process, by keeping the concrete moist, prevents it from shrinking until it has attained greater strength, and thereby minimizes cracking. The daily wetting not only prevents shrinking but actually causes an expansion which seems to be about 0.0001 inch per inch of length; and as long as the wetting continues a compressive stress is set up which must be counterbalanced before the concrete can be subjected to tension.

The importance of this initial expansion lies in the fact that the concrete, from the moment it is placed, is subjected to alternate compressive and tensile stresses caused by increases and decreases in temperature. The compressive stresses are negligible in their effect, but the tensile stresses would often be sufficient to crack the pavement during the early stages were it not for the initial compression set up by the curing process.

CONTRACTION CRACKS CAUSED BY SUBGRADE FRICTION

All contraction cracks, whether they take the form of surface checking or larger cracks extending through the whole depth of the pavement, result from the effort of the contracting concrete to drag itself over a less rapidly contracting portion of the pavement or over the subgrade. In the case of surface checking it is the lower portion of the concrete itself which, opposing the contraction of the surface skin, sets up the tensile forces that crack the surface. The larger cracks result from frictional resistance to the movement of the whole mass of the pavement over the subgrade.

It will be helpful at this point to analyze in detail the action leading up to the formation of one of the larger cracks. We start with a freshly laid concrete pavement. It dries and tends to contract. If it rested upon a perfectly frictionless base the contraction would take place without causing cracking. The pavement, free to move without any resistance, would draw itself together and set up into a solid, uncracked slab. But this perfect condition does not obtain. The subgrade is not a frictionless plane. It offers more or less frictional resistance to the contractive movement of the pavement.

Now, for each condition of dryness or temperature the pavement has a certain normal length which it would assume if unprevented by external forces. If, under any given condition, it is prevented from assuming the normal length pertaining to that condition it is strained, though actually its length may not change. Thus a drying concrete pavement may be increasingly stressed and strained, although its actual length may remain unchanged. It is the normal length that is changing, and as this becomes increasingly less than the actual length the stress mounts up, and a point is finally reached where the tensile strength of the concrete is exceeded and a crack forms. While this process is going on the concrete may not move at all. Its actual length may remain unchanged. If it does move and thereby shorten its actual length, the stress is relieved.

HOW THE CRACKS FORM IN PLAIN CONCRETE PAVEMENTS

With these facts in mind let us now examine the behavior of our drying concrete pavement. It is built without joints. Originally it extends as an unbroken slab of moist concrete between the free ends where the day's work began and ended. As it dries this unbroken slab tends to contract and draw itself over the subgrade from each end toward the center. It is wholly or partially prevented from assuming its normal length, which becomes shorter and shorter, by the frictional resistance of the subgrade, and it is therefore subjected to a tensile stress which increases as it dries.

We may imagine the slab to consist of transverse segments each of which is contracting within itself and each tending to draw its neighbor toward it, and through its neighbor all others to the free end. Movement of each of these segments is resisted by subgrade friction at its base. When, therefore, the initial segment is pulled upon by its neighbor it is held back by the subgrade friction on its base. The second segment, pulled upon by the third, is also held back by forces of friction which are added to those of the first in resisting the movement of the two segments; and similarly the third, fourth, and succeeding segments, each pulls upon the train of segments behind it and is resisted by the combined forces of friction acting upon them.

The pull exerted by any segment is the stress in the concrete at the particular point, and it will be seen from the foregoing that this stress must increase with distance from a free end. It goes on increasing with distance until a point is reached at which it exceeds the tensile strength of the concrete, and at that point a crack occurs.

From the moment the crack forms a new condition is established. There has previously existed a condition of increasing stress from the free end toward the point of cracking. As the stress has increased the deformation has increased. We may imagine that the first segment at the free end has been able to contract practically to its normal length, its contraction being resisted only by its own subgrade friction. The second, however, has had to pull not only against the friction on its own base but against the frictional forces of its neighbor as well. It has, therefore, not been able to assume quite so nearly its normal length as the first; and the third, fourth, and succeeding segments have been increasingly deformed from their normal length. At the point of cracking the deformation is that which results from the tensile breaking stress of the concrete.

But the moment the crack forms this stress is relieved; a new free end is formed, and the concrete near this new end becomes able to assume more nearly its normal length, which it does, thereby opening the crack.

THE CRACKING OF REINFORCED CONCRETE PAVEMENTS

The action that has been described is what takes place when the pavement is not reinforced. If steel is used the action is similar up to the point of cracking, the only difference being that the steel adds its strength to the strength of the concrete in resisting the increasing stress and thus increases slightly the distance between the free end and the first crack in the concrete. But the steel does not crack. It remains as a connection between the dissevered slab and the balance of the pavement. It remains under stress and prevents the concrete in the dissevered slab from assuming its normal length. It keeps the concrete adjacent to the crack in a state of stress nearly equal to that which broke it, and holds the dissevered slab closely to the balance of the pavement, so that the crack in the concrete is not permitted to open. As near the crack the concrete remains in a high state of stress it may be assumed that secondary cracks will readily be formed near the initial crack as a result of bending stresses induced by the passage of vehicles.

Passing from the dissevered slab into the balance of the pavement it enters with an initial tensile stress, which, with respect to the balance of the pavement considered as a free body, becomes an external force which adds itself to the forces of friction resisting the contraction of the concrete and causes the next crack to occur in a lesser distance than the first.

The stress in the steel is increased as it passes through the second slab and continues to increase as it passes through successive slabs until finally the yield point is reached and the steel "necks down" or ruptures. At that point an open crack is formed. This behavior of reinforced concrete was first observed in the experiments of the Bureau of Public Roads on the Columbia Pike experimental road near Washington, D. C., and it has been described in a report published in this magazine.[1]

THE END OF THE CURING PERIOD A CRITICAL TIME

Whenever contraction cracks are formed in a pavement, whether it be in the early stage of its life or at a later priod, the behavior of the pavement prior to cracking is as described above. However, the critical period begins at the end of the curing. The pavement, which previously has been kept in an expanded condition, then begins to contract. The amount of the contraction as measured under laboratory conditions has been found to be as much as 0.0005 inch per inch of length, which is as much as would be caused by a change in temperature of 100° F. This contraction takes place in a period of at least three months under rapid drying conditions. In the field a longer period would undoubtedly be required. But this contraction occurs at a time when the tensile strength of the concrete is lower than at any subsequent period. For this reason contraction cracks are more likely to occur at this period than at any other time; and the strength of the concrete at this period is, therefore, one of the governing factors in any design which aims to prevent the formation of contraction cracks.

Unfortunately the tensile test for concrete is difficult to make and the results are quite variable. It is not possible, therefore, to arrive at a very exact determination of the tensile strength of the concrete during the early periods of its life. For practical purposes it is assumed that the tensile strength is equal to approximately one-twelfth of the compressive strength, and, although it must be granted that there will be considerable variation from them, the values of the tensile strength arrived at in this way are as follows:

Approximate tensile strength of concrete at various early stages

Age (days)	Tensile strength (pounds per square inch)	Age (days)	Tensile strength (pounds per square inch)
7	100	28	250
15	194	90	400

JOINTS AND REINFORCEMENT AS CRACK PREVENTIVES

The use of joints and steel reinforcement are the principal methods employed to prevent the formation of shrinkage cracks. If only the joints are used their effectiveness depends upon their being spaced at intervals slightly shorter than those at which the pavement would naturally crack. When steel is used the fine cracks which form are not generally regarded as objectionable, and the purpose of design may be limited to the prevention of the open cracks that form where the steel is "necked down" or ruptured. Obviously, therefore, the use of steel should be coupled with the use of joints at which the steel is separated, so spaced as to prevent the rupture of the steel and the formation of an open crack.

However, if the steel is entirely separated at the joints it is prevented at these points from acting as a dowel, which is one of its most valuable functions. If, on the other hand, it is not separated in some way it will rupture at intervals and will be no more effective as a dowel than if it is separated at joints. The only means thus far suggested to preserve continuous dowel action is that of greasing the steel, but this method prevents the bars from acting as temperature reinforcement and permits cracks to form as frequently and as wide as in plain concrete.

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[1] Reinforcing and the subgrade as factors in the design of concrete pavements, by J. T. Pauls, PUBLIC ROADS, vol. 5, no. 8, October, 1924.

The method of design and construction which will now be proposed substitutes for the prevailing empirical methods a rational method of spacing the joints, permits the steel to act as temperature reinforcement between joints, and also preserves it valuable dowel action at the joints.

For the determination of the joint spacing it is necessary to know or assume the strength, dimensions, and other properties of the steel and concrete and the coefficient of friction between the pavement and the subgrade. The range of the latter coefficient was determined approximately by tests made by the Bureau of Public Roads several years ago and described in a recent article in PUBLIC ROADS.[2] These tests seem to show that the coefficient of friction varies between the approximate limits of 1.0 and 2.0. No doubt there is a possibility of its exceeding 2.0 under extremely rough subgrade conditions; but for purposes of design under normal conditions a value of 2.0 may be considered as safe and conservative.

THE RATIONAL SPACING OF JOINTS ILLUSTRATED

To illustrate the method employed, let it be assumed that the curing period has terminated at the end of 10 or 15 days. At this time the tensile strength of the concrete probably varies from 120 to 190 pounds per square inch, the probability being that the lower figure is more nearly correct. In that case a factor of safety of 4 would give a safe value of 30 pounds per square inch. If, then, it is desired to find the proper spacing of the joints, such that no cracking will occur between them, it is necessary to equate the total safe tensile resistance of the concrete and steel at the midsection between the joints to the total frictional force between the mid section and a free end (see fig. 1). This condition is expressed by equation 1.

$$f\frac{L}{2}\ WB = 12\ BtS + a\frac{E_s}{E_c}S \quad \text{- - - - - - - - - - - - - - -} \quad (1)$$

in which—

L = Spacing of transverse joints in feet.
B = Width of pavement in feet.
S = Allowable tension in concrete in pounds per square inch.
f = Coefficient of subgrade friction.
W = Weight of concrete in pounds per square foot.
t = Thickness of pavement in inches.
E_s = Modulus of elasticity of steel.
E_c = Modulus of elasticity of concrete.

The required spacing of the joints to satisfy the above condition when various amounts of steel are used is shown in the first column of Table 1. In these computations the values assumed for the various factors other than a are as follows:

$S = 30$ pounds per square inch.
$\frac{E_s}{E_c} = 10.$
$f = 2.0.$
$W = 75$ pounds.
$t = 6$ inches.
$B = 18$ feet.

It will be observed that the use of a rather large amount of steel is of very little value in increasing the spacing of the joints provided no crack is permissible between these joints.

If it is desired to find the spacing such that no open cracks will form, fine cracks being permissible, then it must be assumed that the total tension set up by the force of friction is resisted by the steel, and the joints must be so spaced that the steel will not be stressed to its yield point. This condition is satisfied by equation 2, as follows:

$$f\frac{L}{2}WB = a\ S_1 \quad \text{- - - - - - - - - - - - - - - - - - - -} \quad (2)$$

in which f, L, W, B, and a represent the same factors as in equation 1, and

S_1 = Allowable tension in steel in pounds per square inch.

Assuming values of f, W, and B as in the former example and the value of S_1 as 25,000 pounds per square inch the spacing for various amounts of steel is shown in column 2 of Table 1. From this column it will be seen that if fine cracks are permitted to develop the spacing of the joints may be materially increased by increasing the amount of steel.

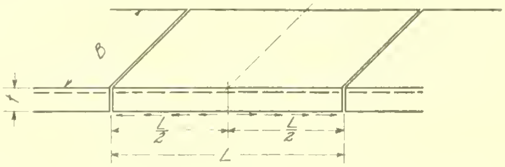

FIG. 1.—Diagram illustrating theory of crack spacing in reinforced concrete pavements

TABLE 1.—*Spacing of transverse joints to prevent (1) all intermediate cracking, and (2) intermediate wide cracks*

Amount of longitudinal steel	Required spacing of transverse joints	
	(1) For no intermediate cracks	(2) For no wide intermediate cracks
	Feet	*Feet*
Plain concrete	28.8	28.8
4¾-inch round bars (a=1.76 square inches)	29.2	32.6
8¾-inch round bars (a=3.52 square inches)	29.6	65.2
12¾-inch round bars (a=5.28 square inches)	30.0	97.8

THE DESIRABILITY OF SMOOTH SUBGRADES AND LONG CURING

It will be observed in equation 1 that L, the spacing of joints, varies directly as the tensile strength of the concrete and inversely as the coefficient of friction at the base. The equation emphasizes the importance of curing the concrete long enough to permit it to attain a relatively high tensile strength before it is allowed to shrink and thus develop tensile stresses. It also shows that the subgrade should be made as smooth as it is practicable to make it in order to keep the force of friction low and thus reduce the tension induced in the concrete when shrinkage takes place.

It is to be acknowledged that the coefficient of friction is the most uncertain of the factors involved in the calculations of the safe joint spacing recorded in Table 1. It is possible that the value of f, assumed to be 2.0 in these calculations, may under some conditions be much less. In that case the joint spacing would be much wider. It is probable that the assumed value of the coefficient is practically the maximum value and the error in the recorded spacing may therefore be regarded as on the safe side. It is possible also that the values assumed for other constants in the equations may be subject to revision as our knowledge increases. It is intended here to suggest a method rather than to determine exact values. It should be possible to obtain a working knowledge of the coefficient of friction that must exist on various subgrades by observation of the intervals at which contraction cracks form in existing pavements on the various soils.

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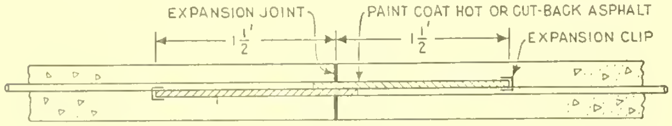
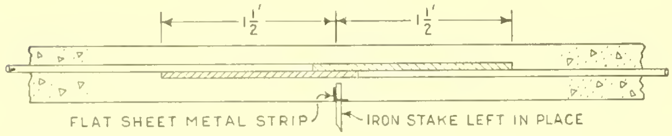
[2] Friction tests of concrete on various subbases, by A. T. Goldbeck, PUBLIC ROAD vol. 5, no. 5, July, 1924.

DOWEL ACTION OF STEEL PRESERVED BY NEW FORM OF JOINT

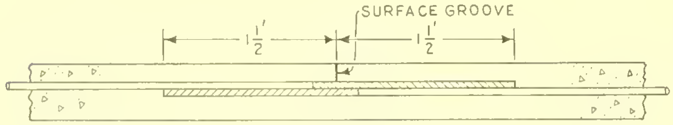
Having determined the spacing of the joints there is the practical problem of how to form them. The method employed in determining the spacing under each of the two conditions examined assumes that a joint will be formed in the steel as well as in the concrete. This is accomplished by the proposed method without sacrifice of the valuable dowel action of the steel at the joints, as shown in Figure 2.



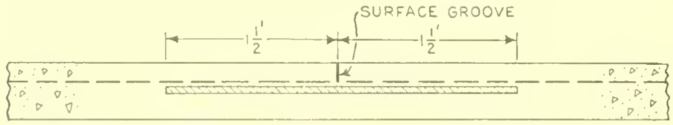
A. METHOD OF FORMING EXPANSION JOINT



B. METHOD OF FORMING CONTRACTION JOINT WITH SUBMERGED METAL STRIP



C. METHOD OF FORMING CONTRACTION JOINT BY GROOVING SURFACE



D. METHOD OF FORMING CONTRACTION JOINT WHEN MESH REINFORCEMENT IS USED

FIG. 2. Methods proposed for forming joints in steel reinforcement and reinforced concrete pavements. The methods permit the contraction of the concrete without rupturing the steel

The desirable dowel action is provided by extending the steel from each slab a distance of several feet into the adjacent slab, and painting the extended ends of the bars with a hot asphaltic material to break the bond with the adjacent slab. If an expansion joint is required the method is as shown in Figure 2A.

However, it will probably not be necessary to provide expansion joints at the frequent intervals indicated in Table 1. At many of these intervals it will be necessary only to provide for contraction. This may be accomplished as shown in Figure 2B by introducing a submerged thin steel plate in such a way as to form a plane of cleavage. In this case no joint will be formed during construction, but a straight, wide crack will form over the submerged plate at the predetermined location. These cracks would need maintenance, but would probably be no more difficult to maintain than a formed joint. The same object might be accomplished by cutting a groove a few inches deep in the surface at the proper stage of hardening as shown in Figure 2C. Finally, if mesh, instead of bar reinforcement is used the same object is attained by the use of asphalted dowels as shown in Figure 2D.

By the use of any of these methods it is assured that the stress in the steel will only be that due to the subgrade friction on a predetermined length of the concrete, and the overlapping of the bars provides dowel action at the joints as well as at other points.

NEW MOTION PICTURE RELEASED

"Crossing the Great Salt Desert" is the title of the latest motion picture on road building which has been prepared for the Bureau of Public Roads by the Office of Motion Pictures, United States Department of Agriculture. The film, which is now ready for distribution, may be obtained by application to the Office of Motion Pictures, United States Department of Agriculture, Washington, D. C. Copies of the film are furnished free except for transportation charges both ways.

"Crossing the Great Salt Desert" is the story of a barrier overcome. It tells the story of the perils and privations of the early settlers who, with their wives and families, struggled in covered wagons across the Great Plains, fought off hostile Indians, traversed snow-covered mountain divides, and finally arrived at the eastern border of the Great Salt Desert.

Depending upon the season they then were forced either to stagger across this arid waste under the blistering rays of a summer sun or to plod wearily over the mud flats or brine-covered salt deposits in the winter. This desert was the terror of the western emigrants. Those who reached the other side in safety usually were successful in completing the westward journey and in reaching the promised land of settlement toward which all their arduous labors had been directed.

Animated views illustrate the northerly transcontinental road followed by the Forty-niners across the Great Salt Desert during the California gold rush. The southern route around and over the southern borders of the desert is shown to have been followed at a later date by Mormon settlers. The selection of one of the transcontinental routes, which forms a part of the Federal Aid Highway System, made necessary a choice between the old north and south roads. The northerly route was selected for the highway because it had the advantage in distance, travel time, snow conditions, grade, and ultimate cost.

The picture is replete with detailed construction views of the 40-mile section across the Great Salt Desert between Wendover and Knolls, Utah, known as the Wendover Cut-off. This was built as a Federal-aid project by the Utah State Highway Department in cooperation with the Bureau of Public Roads. The unique machinery used for making the embankment across the brine-covered salt beds, the difficulties of construction, the hardships of the men and teams, automobiles speeding over the completed roadway, a striking Utah sunset, and many other details, all make this picture of special interest to road builders and to the general public.

A NEW TEST FOR CONSISTENCY OF CONCRETE APPLICABLE TO DRY PAVING MIXTURES

DEVELOPED BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by **F. H. JACKSON**, Engineer of Tests, and **GEORGE WERNER**, Assistant Scientific Aid

THE development of a simple yet reliable method for determining the consistency of cement concrete mixes presents a problem which has never been satisfactorialy solved. Although numerous schemes have been proposed from time to time, only two or three of them have ever been developed so as to be of any practical value. The so-called slump test [1] has been used more extensively than any other and, when properly interpreted, has given fairly good results, especially as a method of field control. Its obvious limitations, however, especially as regards its use for the control of dry mixtures, lean mixtures, and concretes in which large-sized crushed stone is used, have prevented its general adoption. The flow test [2] is used to a certain extent as a method of laboratory control, but the nature of the apparatus required makes it impractical or use in the field. These are the more widely known methods. There is one other that merits serious consideration, namely, the penetration test for "workability" of concrete proposed by Pearson and Hitchcock;[3] and this method would appear to be limited in application to concrete of the consistency usually required in reinforced concrete building work and unsuited to the dry mixes employed in road construction.

There has been a great deal of discussion in recent years as to just what is meant by the terms "consistency," "workability," etc., as applied to concrete mixtures. According to Abrams,[4] consistency may be defined as "the relative plasticity or workability of freshly mixed material." Pearson and Hitchcock, on the other hand, in their paper describing the penetration test, distinguish between consistency and workability, holding that the former term should be used only to describe the condition of the concrete as it is affected by changes in water content, whereas the latter term should be used to describe that condition of a given mixture which depends not only upon the water content, but upon any factor which affects the amount of work required to place and finish the concrete in a satisfactory manner. It is, of course, apparent to anyone that there is a great difference between the "workability" of a rich, high-sanded, 1:2:3 mix and a lean, 1:3:6 mix, even though both mixes may have the same "consistency" as measured by the slump or flow test. Furthermore, it is impossible by any change in the water content of the leaner mixes to secure as workable a mix as may be obtained with the former. From this viewpoint, therefore, it would appear that there is justification for a distinction between the terms. It has often been demonstrated, however, that by far the greatest variation in the strength of concrete in actual practical construction is due to changes in the water content. Granting, therefore, the effect of water on strength, and bearing in mind also that variations in the amount of water required to produce concrete of a given consistency may be caused by any of the factors which affect workability, such as gradation of aggregate, cement content, etc., it would appear that the practical control of the water is the vital thing to be considered. In this paper a new test is described which may be used for controlling the water content and therefore the strength of any given concrete mixture.

The test was designed as a substitute for the slump test and is proposed for use primarily as a method of field control. The process is based upon the principle that the consistency of concrete may be determined by weighing the amount which is retained upon a circular plate of given diameter when the concrete is deposited thereon in any standard manner. The device which has been used by the Bureau of Public Roads in demonstrating this principle is shown in Figure 1.

DESCRIPTION OF THE APPARATUS AND TEST

The apparatus consists essentially of a box spring scale upon which is mounted a circular steel plate 12 inches in diameter. A hopper supported by an angle-iron frame is mounted over the plate, so that the point of discharge is 12 inches from the surface of the steel plate. The hopper, which will hold approximately 45 pounds of wet concrete, is provided at the discharge end with a steel slide, the withdrawal of which permits the concrete to fall upon the plate. The test is made in the following manner. The apparatus is set up adjacent to the point where the batch of concrete to be tested will be deposited, and the hopper, which will hold about 50 per cent more concrete than it is possible to retain upon the 12-inch plate under any circumstances, is filled directly from the pile by means of shovels. It has been found that the exact procedure to be followed in filling the hopper is immaterial so long as it is filled completely and no attempt is made to jostle or compact the concrete. Mounted beneath the steel plate are two cams, operated by a handle, and so arranged that the weight on the plate may be taken off the spring. With the cams set in this position the slide is drawn and the concrete is allowed to flow out upon the plate until the hopper is empty, and when all movement has ceased the handle controlling the cam is turned and the concrete upon the plate is weighed. This weight is taken as the index of the consistency of the concrete. Views of the apparatus with the hopper full and after discharge of a wet and a dry concrete are shown in Figure 1.

THE RELIABILITY OF THE TEST

Numerous tests with this device on concrete of various proportions and with various sizes and types of aggregate indicate that it will truly measure the relative consistency of the mix. In one series of tests, for instance, in which gravel graded up to 1½ inches in size was used in a 1:2:4 mix, the amount of concrete retained upon the plate varied from 21 pounds with a water-cement ratio of 0.85 to 6 pounds with a water-cement ratio of 1.2. In this series of tests which was made some months ago a number of batches of 1:2:4

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[1] U. S. Dept. of Agric. Bul. 949, Standard and Tentative Methods of Sampling and Testing Highway Materials, p. 66.

[2] Proc. of the American Society for Testing Materials, vol. 21, p. 983.

[3] Proc. of the American Society for Testing Materials, vol. 23, pt. 2, p. 276.

[4] Proc. of the American Society for Testing Materials, vol. 23, pt. 2, p. 443.

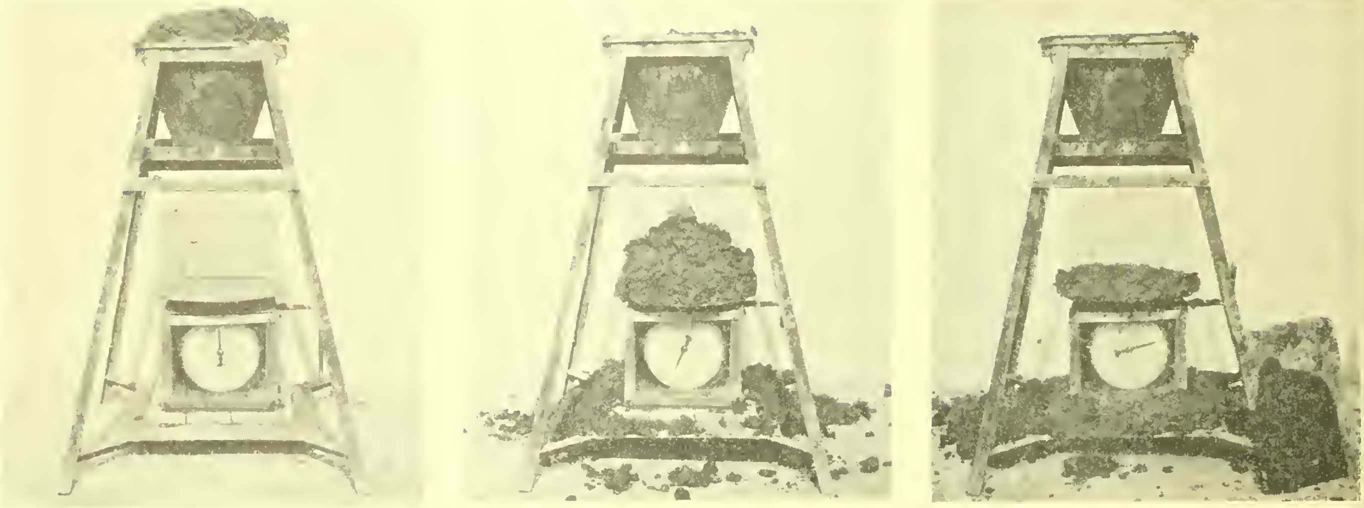

FIG. 1.—Left—the device with hopper full. Center—after discharge of a dry concrete. Right—after discharge of a wet concrete

concrete were made in which the water-cement ratio was increased by increments of 0.05. Slump tests, flow tests, and tests with the 12-inch plate were made on each batch after which the material was cast into 6 by 12-inch cylinders and tested for compressive strength at the age of 28 days. Results of this series of tests are shown in Figure 2. Note the striking similarity between the curve showing the relation between water-cement ratio and crushing strength and the curve for water-cement ratio and consistency as determined by the plate tester. It will be observed that there is a tendency for the amount of concrete retained upon the plate to drop off for very dry consistencies in exactly the same way as the strength falls off. It occured to the authors that if the relationship shown in Figure 2 were true throughout the entire range of working consistencies and for various mixes and types of aggregate that the plate test should give directly a very good indication of the probable relative strength of the concrete. In this particular series the slump test showed up very poorly, although it was made strictly in accordance with the methods described in the A. S. T. M. tentative standards, except that the mold was removed immediately after placing the concrete. It will be noted that the concrete did not slump at all until a water-cement ratio of 1.1 had been reached, corresponding to a flow of 160 and a plate test of 9 pounds. Granted that this is very unusual, these results nevertheless serve to illustrate the uncertainties of the slump test. The flow test results in this series were, on the other hand, very concordant, showing practically a straight line relation for a water-cement ratio varying from 0.75 to 1.2.

Having in mind the interesting possibilities in the way of strength control indicated by this initial series of tests, a somewhat more extensive investigation was begun for the purpose of determining whether these relations held for different types and sizes of aggregates and different proportions. In this series of tests, six different mixes were used: 1:1½:3, 1:2:3, 1:2:3½, 1:2:4, 1:2½:5, and a 1:3:6. Three types of aggregates—Potomac River gravel, crushed limestone, and crushed slag—were employed for each mix. The gravel and slag were graded from one-fourth inch to 1½ inches and the limestone from one-fourth inch to 1½ inches for one lot and one-fourth inch to 2½ inches for another lot. The concrete was mixed by hand with shovels in 1 cubic foot batches; and four 6 by 12 inch compression specimens were made from the concrete in each batch. Three consistencies were used for each combination—dry, medium and

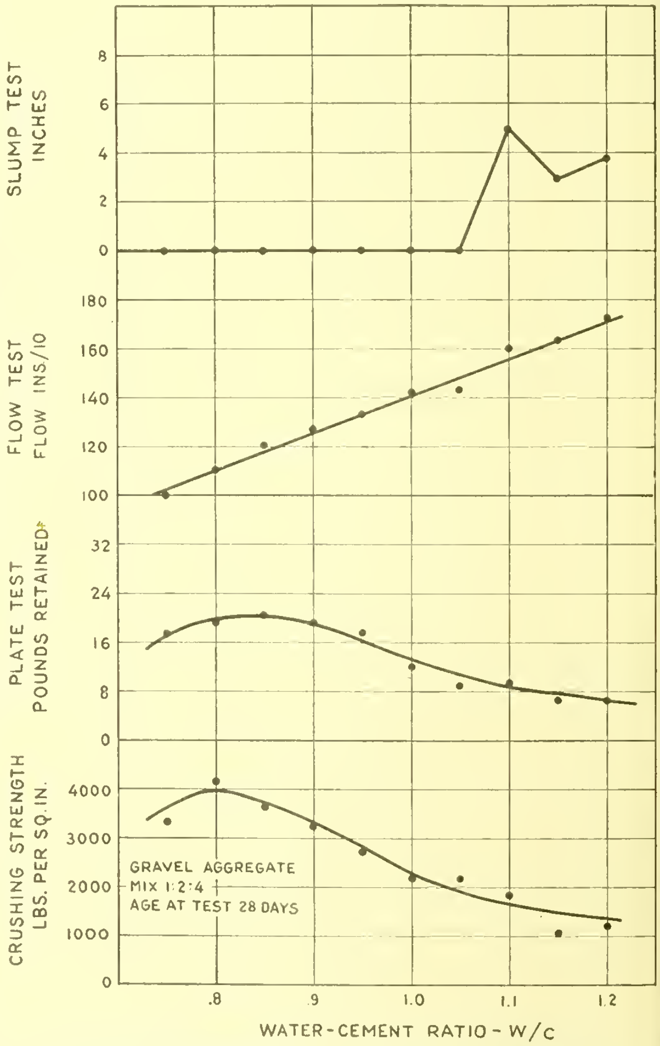

FIG. 2.—Relation of water-cement ratio and strength and corresponding consistency relation as determined by the slump, flow and plate tests

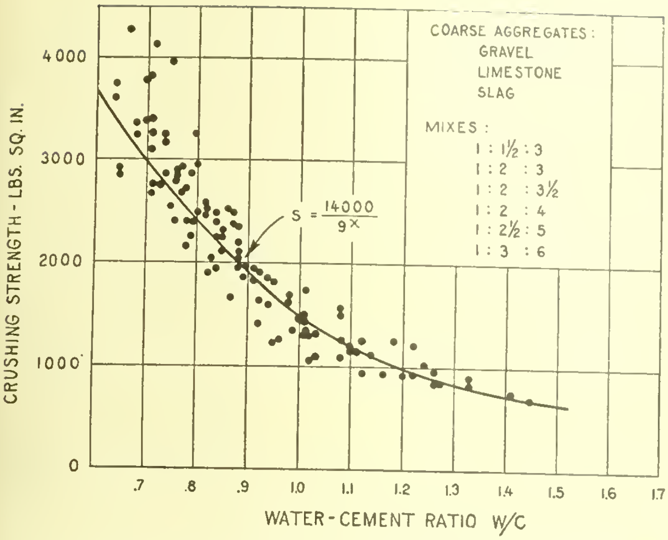

Fig. 3.—Relation of crushing strength and water-cement ratio for all compression tests, showing comparison with the curve suggested by Abrams and Walker

wet: and immediately after mixing, the three tests for consistency—plate test, flow test, and slump test—were made simultaneously on different portions of the batch, by different operators, in order to eliminate any error due to the time element or to rehandling the concrete. Finally, at the conclusion of the first series the entire program was repeated, making a total of approximately 130 separate batches of concrete tested.

COMPARISON WITH OTHER TESTS

In order to check the accuracy of the strength-water-cement-ratio relation, the results of all the compression tests, irrespective of mix, size, or type of aggregate were plotted, and are shown in Figure 3, together with the curve suggested by Abrams and Walker[5] as indicating the strength-water-cement-ratio relation. The equation of this curve is $S = \frac{14{,}000}{9^x}$, in which S equals the crushing strength of the concrete in pounds per square inch at 28 days, and x (an exponent) is the water-cement ratio. It will be seen that the average of all points lie very close to the curve except for the very dry mixes, although it should be noted that these tests were made at the age of 14 instead of 28 days, so that the average of all of the points would probably lie somewhat higher than is here indicated if they had been made at the conventional period.

To determine the relations between the strength and the slump, flow, and plate tests for the different mixes and types of aggregate, the results were plotted separately on a series of charts which are reproduced as Figure 4. It will be observed that the relation between strength and consistency as determined by the plate test is the same, in general, as that shown in Figure 2. Furthermore, the points representing consistency of concrete made with the different coarse aggregates appear with one or two exceptions to lie very much

(Continued on page 136)

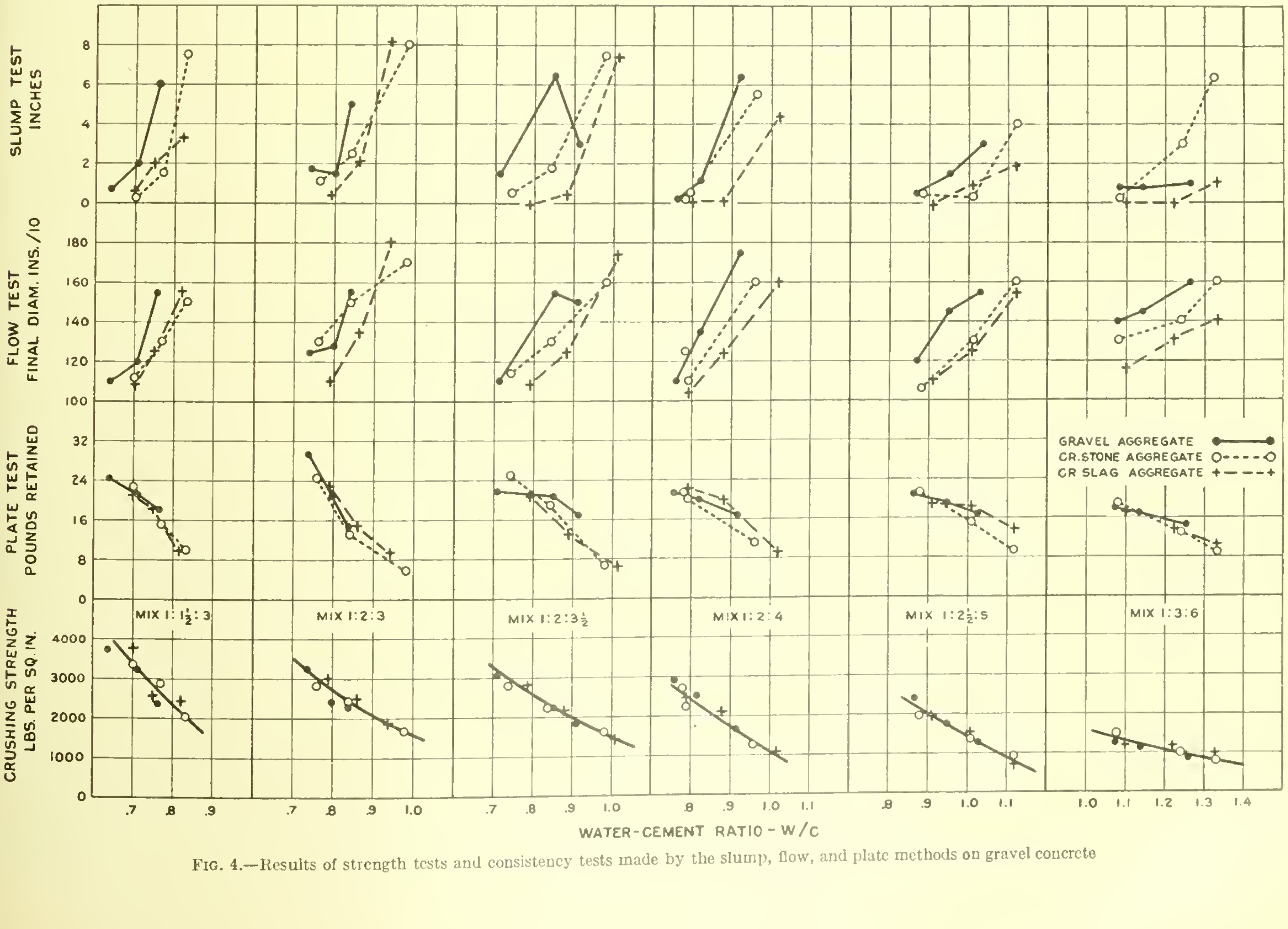

Fig. 4.—Results of strength tests and consistency tests made by the slump, flow, and plate methods on gravel concrete

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[5] Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill., Bul. 9, 2d Ed., April, 1925.

COMMODITY TRANSPORTATION BY MOTOR TRUCK

A SUMMARY OF THE RESULTS OF TRANSPORTATION SURVEYS CONDUCTED BY THE U. S. BUREAU OF PUBLIC ROADS

Reported by J. G. McKAY, Chief, Division of Highway Economics

THE rapid growth of motor-truck transportation during the past few years has resulted from five principal causes, which are: (1) The relative economy of motor-truck transportation with respect to transportation by horse-drawn vehicles; (2) the development of improved rural highways; (3) the demands of shippers for prompt service; (4) the constant improvement in the mechanical efficiency of the vehicle; and (5) the impetus given to motor-truck transportation by the World War.

Mainly as a result of the operation of these causes the registration of trucks in the United States increased from 136,000 in 1915 to 2,130,000 in 1924, an increase of over 1,400 per cent. Although the truck traffic on rural highways has apparently increased at a somewhat slower rate than the registration, it has already attained a volume which makes it a very important part of our system of distribution. As an illustration of the relative growth of traffic and registration it has been found that in Maine motor-truck traffic increased approximately 500 per cent from 1916 to 1924, while the registration increased approximately 850 per cent. As an indication of the magnitude of this modern development it may be noted that the transportation of commodities over the Connecticut highway system during the year beginning in September, 1922, reached a total of 88,000,000 ton-miles, and that in Cook County, Ill., during the summer months of 1924 the average daily movement was 61,000 net ton-miles.

The motor trucks used on rural highways are predominantly trucks of small capacity. Results of the highway transportation surveys in the States of Maine, Pennsylvania, and Connecticut, and in Cook County, Ill., indicate that trucks of ½ to 2½ tons capacity constitute from 77.3 per cent in Cook County to 96.8 per cent in Maine of the total number of trucks operating on the rural highways. Trucks of 5 tons capacity, or greater, constitute from 0.3 per cent in Maine to 11.7 per cent in Connecticut of the total number found on the rural highways. Near large cities more heavy-duty trucks operate on the highways, but the truck of large capacity does not travel far from centers of population in the movement of goods.

TRUCK HAULAGE PREDOMINANTLY A LOCAL MOVEMENT IN ALL AREAS

The bulk of the motor-truck movement is local, short-haul transportation. There is some variation in the length of haul depending upon: (1) The prevailing production of the area; (2) shipping distances to market; (3) the type of highway improvement; (4) the other types of transportation available; and (5) the distance between centers of population. The comparison of truck haulage in Connecticut, Maine, California, and Cook County, Ill., to be found in Table 1 indicates clearly the predominance of the short-haul movement.

The principal movement of loaded trucks occurs within a zone of 29 miles or less in Connecticut, California, Maine, and Cook County. In Connecticut 79.5 per cent, in California 60.7 per cent, in Maine 80.5 per cent, and in Cook County, Ill., 75.8 per cent of the loaded trucks move less than 30 miles, while in the latter case 55.3 per cent operate less than 20 miles. In Connecticut and Maine 47 per cent move less than 10 miles. Only a small percentage of the truck movement exceeds 60 miles; in Connecticut 7.9 per cent, Maine 6.6 per cent, and in Cook County, Ill., 5.4 per cent.

TABLE 1.—*The percentage of loaded trucks operating various distances in several areas*

Length of haul (miles)	Percentage of loaded trucks, Connecticut, 1922–1923	Percentage of loaded trucks, California, 1922 [1]	Percentage of loaded trucks, Maine, 1924	Percentage of loaded trucks, Cook County, Ill., 1924
	Per cent	*Per cent*	*Per cent*	*Per cent*
0–9	47.7	26.8	47.0	24.3
10–19	21.3	22.2	23.1	31.0
20–29	10.5	11.7	10.4	20.5
30–39	6.9	8.3	6.9	9.4
40–49	2.9	5.5	4.1	6.6
50–59	2.8	6.5	1.9	2.8
60–69	1.3	4.7	1.2	1.1
70–79	1.5		.8	.6
80–89	.5	3.1	.7	.4
90–99	.5		.5	.5
100 and over	4.1	11.2	3.4	2.8

[1] California data limited to important highways upon which the percentage of long-haul trucking is greater than upon all highways in the State.

A more exact analysis, based on the net tonnage hauled in the various mileage zones in Connecticut, is presented in Table 2.

TABLE 2.—*Percentage of the total net tonnage hauled in Connecticut by mileage zones*

	Mileage zones											
	0–9	10–19	20–29	30–39	40–49	50–59	60–69	70–79	80–89	90–99	100 and over	Total
	Per cent of total net tonnage											
Products of agriculture	2.8	1.7	0.9	0.5	0.2	0.4	0.1	0.2	0.0	0.1	0.3	7.2
Products of animals	2.9	2.1	1.3	1.0	.6	.4	.2	.4	.1	.1	.3	9.4
Products of mines	7.5	1.4	.2	.2	.1	.1	.1	.0	.0	.0	.0	9.6
Products of forests	2.3	.8	.6	.3	.1	.1	.1	.0	.1	.0	.4	4.8
Products of manufactures	20.8	13.2	8.6	7.1	3.0	3.6	1.7	2.0	.6	.8	7.6	69.0
Total	36.3	19.2	11.6	9.1	4.0	4.6	2.2	2.6	.8	1.0	8.6	100.0

Of Connecticut's total net tonnage, as shown by the table, 67.1 per cent is hauled 29 miles or less and 15.2 per cent is hauled 60 miles or more. Household goods constitute a considerable portion of the goods hauled long distances in this State. Of the total tonnage, 7.2 per cent is composed of products of agriculture, 9.4 per cent products of animals, 9.6 per cent products of mines, 4.8 per cent products of forests, and 69.0 per cent are manufactured products. Manufactured goods are hauled over longer distances than goods of the other principal commodity groups, the large percentage reflecting the type of production in the State. The movement of products of mines is restricted largely to distances less than 10 miles and is primarily a movement of sand, gravel, stone and coal.

THE MOTOR-TRUCK MOVEMENT LARGELY A DISTRIBUTING SERVICE

A large percentage of motor-truck transportation is a direct distribution of commodities to points of final use. In Cook County, Ill., approximately 70 per cent of all loaded trucks were found to be engaged in a service of this character. Analyzed according to destination 25.8 per cent of the loaded trucks were found to be hauling to retail establishments, 17.7 per cent to consumers, 20.8 per cent to construction and repair jobs, 5.5 per cent to farms, and 30.2 per cent to other destinations. Of these several groups all but the last mentioned were engaged in a service of direct distribution of consumption goods or in delivering goods to the point of final use.

Food products and construction materials are the two principal types of commodities hauled by motor truck. In Cook County the analysis of commodities hauled resulted in the following classification:

Commodity	Per cent of net tonnage
Fruits and vegetables	8.0
Miscellaneous products of agriculture	1.4
Milk and cream	4.8
Other dairy products	.4
Meats and packing-house products	1.7
Miscellaneous products of animals	2.4
Bakery goods	1.4
Groceries	4.9
All other commodities	75.0
	100.0

As will be seen from the foregoing, the principal food products make up 25 per cent of the net tonnage. Miscellaneous food products, which are not individually important but which in the aggregate constitute a considerable tonnage, will increase this percentage. Approximately 32 per cent of the daily milk supply of Chicago is brought into the city by motor truck, and the greater part of the milk shipped to the city from points within a radius of 50 miles is trucked in.

Commodities hauled by truck in a given area reflect largely the production in the area, the size and distribution of centers of population and to a limited extent the adequacy of the service of other means of transportation available for the movement of commodities.

COMMERCIAL TRUCK HAULING KEENLY COMPETITIVE

The surveys made in the several areas show definitely that the bulk of the truck movement consists of the transportation of goods in trucks belonging to the owner of the goods. The strictly commercial trucker hauls but a small percentage of the total tonnage on rural highways, and this condition is likely to continue.

There is keen competition between the commercial motor truckers. This has resulted in losses in some cases and in a small margin of profit in others, and has caused a rapid turnover of the companies. A knowledge of operating costs is needed to measure the efficiency of any business, yet the commercial trucking companies know very little of their costs. A few of the larger ones with the better organization do keep cost records, but these records as well as the analysis and application of them as a control in management can be considerably improved. The creation of a depreciation fund to replace worn-out equipment is practically unknown to the smaller companies.

There is a distinct field for the commercial trucker; but successful operation and reliable service will depend largely upon:

1. The keeping of reasonably accurate records of the cost of performing service as a guide to operation.
2. Reasonable regulation to protect responsible operators from unfair competition.

As a result of the application of these two principles the following benefits will result:

1. Analysis of cost records will determine profitable and unprofitable business.
2. Commercial haulage rates can be accurately made.
3. The most economical type of transportation in the various mileage zones can be determined.
4. The proper operating equipment and size of equipment can be selected to serve the needs of the communities.
5. The business will be made more stable and secure, which will be of material advantage to users, and operators will be able to perform more satisfactory and efficient service and earn a fair return on their investment.

CLASSES OF MOTOR TRUCK TRANSPORTATION

The general field of motor-truck transportation can be divided into three major parts:

1. Local distribution of commodities.
2. Organized motor truck transportation supplementing existing rail and water service.
3. Long-haul truck transportation of special commodities.

Local distribution of commodities.—This class of transportation constitutes the bulk of the motor-truck tonnage movement. This movement is not strictly competitive with rail or water service. It consists primarily of the distribution of goods within cities and their suburban areas, and between cities and tributary areas.

There are two distinct types of commodity distribution: (1) The distribution of goods having both origin and destination within the local area, such as the wholesale to retail movement, retail to consumer, distribution of building materials, and the delivery of milk, garden truck, fruits and vegetables from rural areas to the cities; and (2) the completion by truck of the transportation service provided by rail and water facilities.

Local distribution will continue to be the bulk of motor-truck transportation and will be handled largely by the owners of the goods themselves, a small portion by the commercial trucker. In either case efficient distribution requires that accurate cost records be maintained. The merchant with an efficient distribution system has a distinct advantage in competition; the commercial trucker who knows his service costs can distinguish between profitable and unprofitable business and raise his profits accordingly. Operating without knowledge of the cost of doing business he can not determine unprofitable business and is usually a marginal or submarginal operator.

A more efficient use of terminal facilities requires close coordination of motor-truck transportation with rail and water facilities. A well-organized motor trucking company, operating in cooperation and preferably controlled by the major rail carriers in the area, would probably be the most efficient operating organization. Ownership of the terminal motor trucking company is not essential, but control by the rail and water lines is essential to guarantee service and becomes imperative if through rates for combined rail, water, and truck transportation are established.

As a major field for the development of motor-truck transportation this movement is limited to large centers of population, primarily the terminal areas.

Service supplementary to rail transportation.—Organized motor-truck service supplementing existing rail transportation offers three types of service.

1. The extension of freight service into areas without rail service.

2. The substitution of motor-truck service for rail service on unprofitable branch rail lines.

3. The combination of rail and truck service for short-haul package freight in terminal areas and on congested trunk lines. The development of this service is illustrated by the Pennsylvania and New York Central truck haulage of package freight. Prompt and rapid service and partial elimination of way-freight trains on heavy-traffic sections is largely responsible for this development. The factor of increased safety of train operation by the partial elimination of the package-freight train is a real advantage.

There is a real service to be performed by extension of motor-truck lines into areas without rail facilities. In such areas the development of motor-truck lines is perhaps the only type of modern transportation capable of performing service since it is becoming more difficult and hazardous to obtain capital for the extension of rail lines into new areas.

The substitution of motor-truck service for rail service on unprofitable branch lines represents economy in providing transportation service as a whole.

The combination truck-and-rail service for short-haul package freight on rail trunk lines demands a close coordination of service. Haulage of rail package freight by motor trucking companies on a contractual basis represents the first step in this development. If this new type of service is economically sound the carrier ultimately should own and operate the motor-truck equipment. Until it demonstrates its place in the scheme of transportation it is natural for the rail lines to shift the risk of ownership of equipment and a large share of operating risks to motor trucking companies. It is to be remembered also that the possibilities of growth of combined rail-and-truck transportation are largely limited to the comparatively few congested terminal areas and the heavy-traffic rail mileage. Its development depends largely upon the legalization of through rates to include the motor truck pick-up-and-delivery service.

The competition of motor trucking companies with rail lines is not economical, assuming that rail operation offers satisfactory service.

Long-haul transportation of special commodities.—This class of service by motor truck is economically sound when speed of delivery, or the avoidance of special packing and crating are the primary considerations. It is also justified for the transportation of perishable commodities. As for the balance of the long-haul movement it is very questionable, assuming efficient rail service, that it is economically sound, or that it is profitable to the motor-trucking company specializing in the long-haul field. Maintenance of cost records and a fair charge for the right to perform this service will determine whether or not this movement is economically sound. Perhaps the principal value of the long-haul trucker as a competitor or potential competitor has been to call the attention of rail management to the shippers' need for rapid and efficient handling of package freight.

Regulation of commercial trucking organizations is essential because of the disastrous effect of unregulated and destructive competition between motor trucking companies in a field in which the demand for service is relatively inelastic and the supply of service is elastic.

Interstate trucking is largely a local distribution affected by nearness to State lines. The balance of the interstate movement consists of long-distance haulage and is a very small part of the total. Regulation of interstate motor truck transportation means practically the regulation of local trucking and is therefore largely a problem of State regulation.

Jurisdictional disputes concerning the regulation of motor trucks and busses operating across State lines have already developed between the State agencies responsible for the regulation of this new transportation business. A national advisory body to adjust problems of regulation between the States may be of value. It is difficult to visualize the administrative ability of a national regulatory body capable of satisfactorily handling the multitude of cases that would naturally develop if complete control and regulation is vested solely in a national agency. If authority to control this new transportation agency is vested in a national agency, delegation of authority by the national control agency to State regulatory agencies, with power to make decisions, subject to final review on appeal to the national agency would possibly be of real service.

UNIFORM ROAD SIGNS ADOPTED

The Joint Board on Interstate Highways meeting in Washington, D. C., Aug. 3–4, adopted a system of interstate roads and a series of standard danger, caution, direction, and informational signs which it will recommend for use in marking and signing the systems selected.

The system adopted will now be mapped, and as promptly as possible a full report will be made to the Secretary of Agriculture, by whom the board was appointed last February.

The selected routes will be numbered in accordance with a system to be evolved by a subcommittee of the board.

The standard route marker will be a typical United States shield painted white, on which will appear in black the name of the appropriate State, the initials U. S., and the route number. If possible the route numbers will be limited to two digits for easy reading, and steps will be taken to prevent the use of the standard marker for any purpose other than for the marking of the selected system of interstate roads.

Other standard signs for grade crossings, curves, steep grades, etc., were adopted and full details as to design, character, and size of lettering and color worked out.

TAR PAPER ON LOESS SUBGRADE LESSENS HAIR CRACKS IN CONCRETE PAVEMENT

By R. W. CRUM, Engineer of Materials and Tests, Iowa State Highway Commission

HAIR cracks developed during the curing period on the first cement concrete pavement laid on the loess soil in the western part of the State by the Iowa State Highway Commission. This pavement extended through the hilly country adjacent to the flood plain of the Missouri River. The cracks or checks were of the kind usually called hair cracks, although they were generally longer and wider than those commonly encountered in patches on many concrete pavements. They were not surface cracks only. On the contrary, they extended through the pavement to the bottom of the slab.

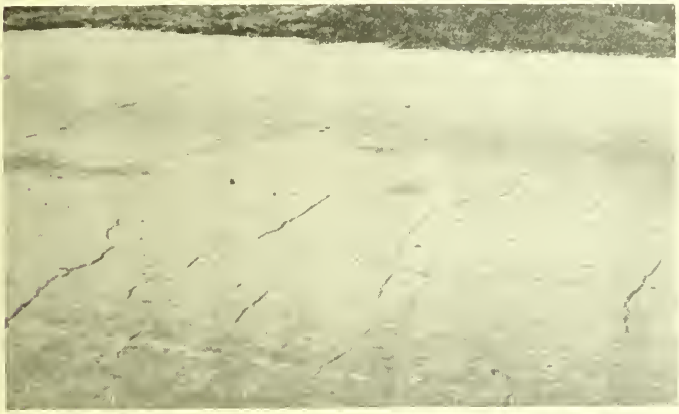

FIG. 1.—Curved formation of hair cracks which is the typical condition when concrete pavement is laid downhill

Careful investigation of the construction conditions, together with some corroborative experiments, indicated that the cracking occurred when the loess soil was dry or slightly moist and most frequently when the pavement was being laid downhill. Upon later work this cracking was eliminated to some extent by thoroughly wetting the subgrade immediately prior to pouring the concrete. A much more effective method, however, was found to be the placing of a layer of tar paper on the subgrade before laying the pavement. This tar-paper treatment has been found to be especially successful and is now the standard practice for concrete pavements to be laid on this type of Iowa soil.

The use of tar paper is not suggested as a cure for all hair cracking. Other conditions than the character of the soil are doubtless often responsible. Apparently the tar paper does not decrease the amount of transverse and longitudinal cracking which develops after the pavement is built and opened to traffic, but definite conclusions can not be given concerning this phase of the matter because it has not been studied in detail.

DESCRIPTION OF HAIR CRACKING ON FIRST CONCRETE PAVEMENT

The data in Figure 3 describe the conditions under which this first concrete pavement was laid and the character of the cracking which developed. The significant conditions upon which the hair cracking depends may be observed on this typical section of the road. The subgrade was uniformly loess soil, hard and smooth, which had been lightly sprinkled just before the concrete was placed. Two brands of cement were used and changed at frequent intervals but this had no apparent effect upon the cracking.

Beginning at station 578 at the right of Figure 3 and proceeding toward the left, in the order that the concrete was placed, it will be noticed that for about 200 feet a large number of cracks appear and this is where the concrete was placed downhill (see fig. 1). Across the sag a few cracks may also be seen, but going up the other side there is none until the crest is reached. From there on, down to the foot of the grade, cracks are prevalent, but again across the bottom and up the hill on the other side only a few are found. Once more, as soon as the crest is passed they appear again until the close of work on August 20 at about station 523.

August 21, being Sunday, no work was done, and on August 22 a heavy rain occurred so that when work was resumed on August 23 the subgrade was in a thoroughly saturated condition. Even though the operation was still downgrade, no hair cracks appeared in the concrete laid immediately after the rain. But they appear again in that which was laid several days later when the subgrade had dried out. These phenomena were repeated several times in the 17 miles of pavement placed that year. Only three remedies suggested themselves—either the concrete would have to be always laid uphill (which, of course, is impracticable); or the subgrade would have to be saturated; or possibly an impervious layer might be placed between the subgrade and the concrete.

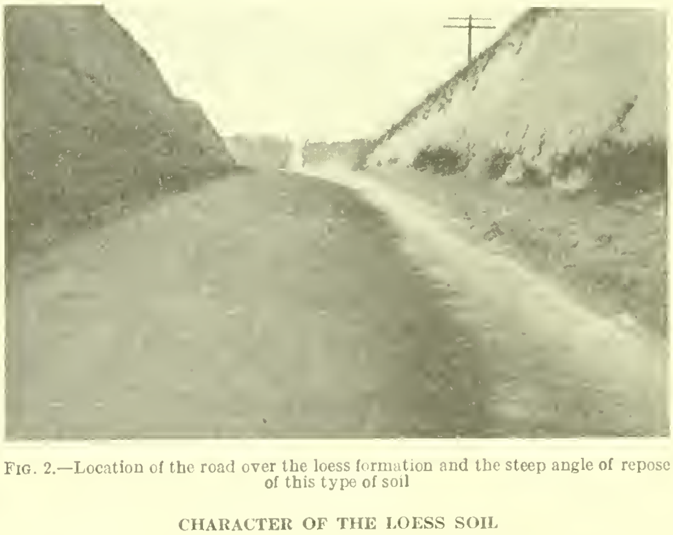

FIG. 2.—Location of the road over the loess formation and the steep angle of repose of this type of soil

CHARACTER OF THE LOESS SOIL

According to most authorities on Iowa geology the loess soil was deposited by wind over glacial drift. It extends back from the Missouri River for a distance of 50 miles or more and forms a mantle so thick that few highway cuts are deep enough to penetrate it. It is a very finely divided material which is eroded rapidly by running water, yet it will stand on a slope so steep as to be almost vertical. In recognition of these conditions it has become the standard practice

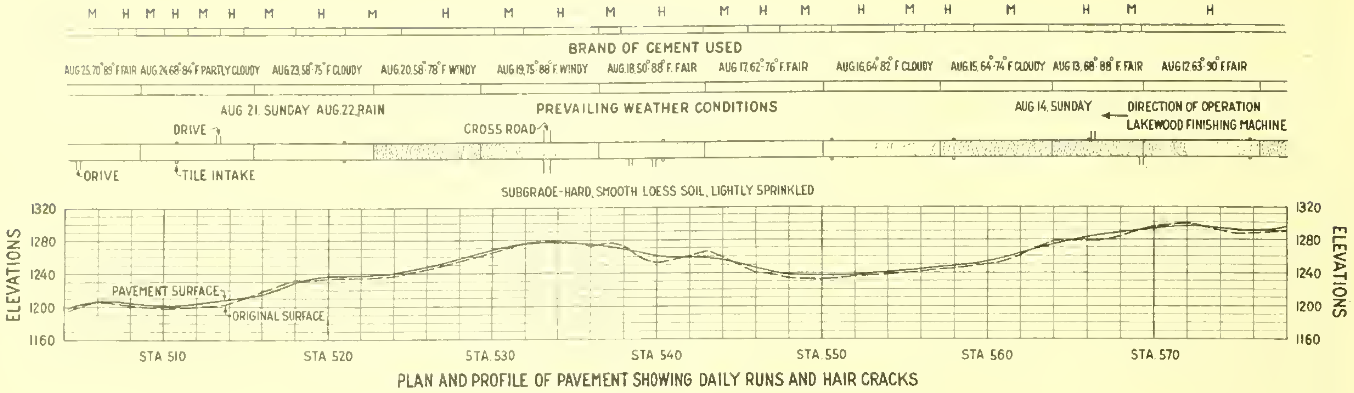

FIG. 3.—Typical section of first concrete pavement laid on loess soil of western Iowa. Hair cracking is most prevalent on portion of the pavement laid downhill

to excavate cuts to a practically vertical slope (fig. 2) and to carry surface water over the pavement, which is built with a curbed section, rather than in side ditches (fig. 4).

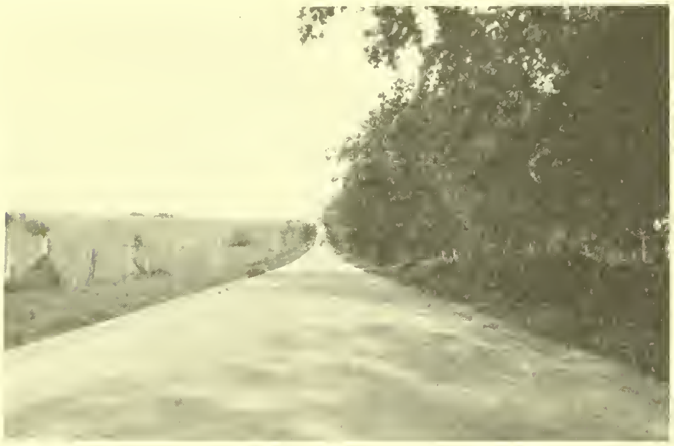

FIG. 4.—Topography of loess soil area and curbed section of concrete pavement

Typical physical characteristics of the soil, determined by the methods of test described in the April, 1925, issue of PUBLIC ROADS are as follows:

		Per cent
Mechanical analysis	Sand	0.74
	Silt	67.08
	Clay	32.10
Moisture equivalent		21.9
Capillary moisture		43.7
Lineal shrinkage (by test of 1 by 1 by 10 inch bar)		2.30

LABORATORY TESTS INDICATE MOISTURE ABSORPTION AS CAUSE OF CRACKING

Some corroborative studies were made in the laboratory to determine the cause of the hair cracks in the pavement laid on the loess soil. The studies were made on mortar slabs, 18 inches wide and 2 inches thick, the proportions being 1 part cement to 2.17 parts of sand by weight.

In these experiments the following phenomena were noted:

1. Slabs on dry or damp loess or yellow clay cracked badly within three hours after being placed.
2. Slabs on saturated clay or loess did not crack.
3. Slabs laid on tar paper did not crack.
4. Dry mixtures showed much less tendency to crack than wet mixtures.

PRESENT IOWA PRACTICE

In view of the foregoing observations, on the next contract the contractor was given the option of using tar paper or thoroughly wetting the subgrade for the length of one day's run ahead of the mixer. He chose the additional wetting, and there was decided improvement, although numerous hair cracks still developed. This method required a great deal of water, and when the subgrade was soaked enough to insure against the cracking it was really too muddy for the trucks to operate properly.

On later contracts the tar paper has been required and the troublesome hair cracks have been practically eliminated. On 14 miles completed in 1924 in a location similar in every way to that shown in Figure 3 only two small patches of cracks have been noted, and it is known definitely in one of these cases that the tar paper was omitted.

HIGHWAY RESEARCH BOARD TO REPORT ON REINFORCED CONCRETE ROADS

A report presented at a recent meeting of the executive committee of the Highway Research Board by C. A. Hogentogler, chairman of the special investigation of the economic value of steel reinforcement in concrete roads, shows that inspections have been completed on 375 miles of plain and reinforced concrete roads, varying in age from 1 to 10 years, and containing approximately 300 comparisons of slabs with and without steel which were subjected to the same influencing conditions. These roads are located in Massachusetts, Connecticut, New York, New Jersey, Pennsylvania, Delaware, Maryland, Virginia, North Carolina, Georgia, and Ohio. The States still listed for inspection are Michigan, Illinois, Missouri, Wisconsin, Iowa, Utah, Washington, California, Texas, and Mississippi.

It is thus assured that a sufficient number of direct comparisons will be made to warrant drawing definite conclusions regarding the effect of steel as influenced by age, design, traffic, climate, and subgrade, as well as by the type, weight, and placement of the reinforcement itself. Maintenance costs of both plain and reinforced concrete roads have also been secured on a large mileage. These costs are in such detail as to show how the maintenance is influenced by any desired variable.

The final report on this investigation will be one of the most complete studies of concrete roads ever undertaken. It will be presented at the fifth annual meeting of the Highway Research Board to be held in Washington, D. C., on December 3 and 4, 1925.

PRESENT STATUS OF THE TRUCK TIRE TESTS OF THE BUREAU OF PUBLIC ROADS

A PRELIMINARY REPORT

BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by E. B. SMITH, Engineer of Tests

DURING the past two years the Bureau of Public Roads has been carrying on a series of tests to determine the relative cushioning properties of solid rubber, cushion, and pneumatic truck tires. This information is of vital interest to truck manufacturers because it may suggest a method for reducing the force of the shocks transmitted to the mechanism and thereby lengthen the life of the truck. The truck user is also benefited by any improvement which tends to decrease the annual operating costs of the truck. Truck manufacturers and users, however, representing only a small fraction of the population, are not the only beneficiaries, since the tests also provide data relative to the destructive effect of the different types of tires on the road surfaces which are built at the expense of the public.

WHEN AND BY WHOM TESTS WERE INITIATED

The tests were initiated about two years ago by representatives of the Bureau of Public Roads in conference with representatives of the various tire and truck manufacturers. A cooperative committee was chosen to prepare a preliminary outline of a series of tests which would furnish useful information relative to tires, trucks, and road surfaces. This cooperative committee represented the Bureau of Public Roads, the Society of Automotive Engineers, and the Rubber Association of America. The rather extensive series of tests which were outlined included the use of different trucks, most of the representative tires of different types and sizes (necessitating several hundred tire changes), and the constant employment of 8 to 10 men. It was expected that the series would extend over a period of more than three years.

The Bureau of Public Roads is conducting these tests at the Arlington Experiment Station, near Washington, D. C., and is providing all the personnel, testing facilities, apparatus, and instruments. The other cooperating agencies are furnishing all the tires, extra wheels, and trucks. After the tests were begun many unforeseen duplications were eliminated and the time of the investigation was considerably reduced. Present estimates indicate that this series of tests, which has been in progress less than two years, will be completed probably next fall, at least in so far as the experimental work is concerned.

In stating the facts to be determined by these investigations no one object can be selected as the most important. The principal factors involved are of course the cushioning qualities of various types of tires and the relative impact they deliver to road surfaces. However, these primary factors are influenced by variations in the sprung and unsprung loads, the speed, the temperature, the condition and kind of tire, and the roughness of the road surface. These variations necessitate principally tests under impact conditinos, but for each tire a static-load test is also made. In securing the data sufficient information is also obtained to determine the force delivered to road surfaces by both impact and static loads. It is possible that a careful compilation of the data will reveal a short and accurate test for classifying tires.

Although these tests are not finished, some of the preliminary indications have been already of real service to the automotive industry and improvements have resulted. Furthermore, the series is developing information which will be of value in the formulation of traffic rules and regulations.

TREND OF RESULTS

At this time it is not proper to publish any of the detailed numerical data which have been obtained, although it should be of interest, in a general way, to indicate the trend of the results.

It is found that the magnitude of the deceleration of a truck wheel may be as high as 2,500 feet per second, as in the case of a badly worn tire. In some cases this may produce an impact force equivalent to 15 times the static load. This, however, is a very extreme condition rarely found in actual practice. It corresponds to a truck carrying a load of 5 tons and with little or no rubber on the steel wheel rim.

Wide tires give higher impact forces than narrow tires for the same load. This does not mean that narrow tires should be used in preference to wide tires in order to prevent the destruction of road surfaces. On the contrary, tires should not be reduced in width beyond a load of 800 pounds per inch of width for wide tires and 600 pounds per inch of width for smaller tires (less than 5 inches in width) on account of the possibility of causing ruts in rock and asphalt roads and the further possibility of reducing the mileage service of the tire.

The vertical thickness of rubber available for cushioning in solid and cushion tires is a very important factor. An ordinarily worn tire may deliver an impact blow as great as seven times the static load of the same tire.

An unevenly worn or damaged tire may easily produce a force equivalent to that of a very rough road both in the impact the road surface will be subjected to and the equal and opposite shocks which will be transmitted to the truck.

On smooth pavements it is self-evident that the cushioning qualities of tires and the impact forces developed by traffic are of small value and consequence and not capable of being measured. Only on rough and wavy pavements do these qualities and force factors become of sufficient magnitude to be measured.

BRIEF DESCRIPTION OF IMPACT TESTS

The impact test consists in operating a truck equipped with test tires over representative sections of smooth concrete and rough granite-block roads, and over a series of artificial obstructions securely anchored to another section of smooth concrete road.

These artificial obstructions consist of inclined planes, rectangular and rounded blocks of varying heights. The impact is determined both on striking these obstructions and at the instant the wheel hits the ground on the first rebound. Sufficient variations in speed are made to determine a reliable curve and the truck load is varied from a condition of emptiness to such an amount that the tires will be overloaded 50 per cent. The determination of the total impact force is made by measuring the deceleration of a rear wheel and the deflection of the corresponding truck spring at the instant of impact.

The deceleration of the rear wheel is measured by a specially designed accelerometer. This instrument is mounted on the side of the truck body, although it obtains its motion through a rigid vertical rod extending down and connecting to the hub cap. The connection to the hub cap is made by means of a specially constructed, tight-fitting universal ball bearing. The deceleration is measured with this accelerometer by virtue of a small weight supported on a calibrated spring. A record is made on the moving paper tape by means of a pencil attached to the decelerometer weight. The vertical height of the record indicates the deceleration according to a scale, which depends upon the spring used under the accelerometer weight; and the cushioning qualities of the tire under test are indicated by the vertical height of the record. The horizontal scale of the record represents the position on the road. In this way the acceleration of the truck wheel may be accurately and definitely located on the record for any position on the road surface. On this same paper tape a record is also made by means of a special attachment between the rear axle and the body showing the deflection of the truck spring at any instant. Before testing, all truck springs are removed, cleaned, and greased, and are kept in this condition by being covered with a canvas boot. These springs are carefully calibrated for load and deflection under a testing machine. The total force delivered to the road surface is then the impact as determined from the deceleration and mass of the rear wheel (unsprung weight) plus the sprung weight as determined by the position of the truck spring at the same instant.

RELATION OF IMPACT TO STATIC LOADS

Each tire was subjected to a static-load test in a testing machine in which the load and deflection were recorded for several conditions of loading up to about 500 per cent overload and tire imprints or areas of contact were obtained. Although there is sometimes an indication of a relation between static and impact tests, a definite relation has not yet been established which can be applied to all tires for the purpose of predicting from the results of the static tests what the action will be under different conditions of impact.

The cushioning quality of a tire is measured by means of the accelerometer and has a direct relation to the deceleration resulting when the tire strikes the road surface with a given velocity. Undoubtedly the tire which offers the best cushioning qualities is one which will give the lowest deceleration value under the condition of impact. In other words, the tire which will cause the wheel to come to rest (in its vertical motion) in a greater distance will have a smaller deceleration value and consequently better cushioning qualities. These cushioning qualities of the different tires are dependent not only upon their inherent characteristics, but also to some extent on the character of the obstruction which they may strike. The horizontal speed with which the truck wheel strikes an obstruction or irregularity in the road surface and the load which it carries are also influencing factors. The cushioning quality of any particular tire is affected also to some extent by the tire temperature.

The cushioning quality of the tires as influenced by the tire temperature was carefully determined by mounting the tire on a truck wheel and placing it in a special impact machine wherein the tire was raised and dropped in a vertical position from different heights. The resulting deceleration was measured with an accelerometer. The temperatures were obtained by means of a water bath and electric heating units. The tire temperature was measured by means of thermocouples thrust into the body of the rubber composition, the temperature being varied from about 30° F. to 210° F. The results of these tests showed that the cushioning qualities were greater for hot tires than for cold tires and that the variation of cushioning with respect to the temperature was rectilinear.

CHARACTER OF ROAD SURFACES

The road runs of these truck-tire tests were always over the same selected stretches of different types of roads and, as far as possible, the truck wheels always followed the same line on the pavement. This resulted in a very good comparison. The two principal types of road surface used were a reasonably smooth concrete and a fairly rough granite (Belgian) block pavement. At the beginning of the tests a bituminous concrete road section was also used, but it was discovered in a short time that this changed in surface roughness to such an extent that successive comparisons could not be depended upon, and it was therefore eliminated.

The different types and sizes of trucks used in this series of tests included a 2-ton worm-drive Mack, a 3-ton Autocar, a 1-ton Ford with special gear reduction, a standard 5-ton class B Army truck, and a 6-wheel type 5-ton Army truck. These trucks all had additional wheel equipment whereby most of them could be equipped with solid, cushion, or pneumatic tires. The speed of operation varied from about 3 miles an hour to the maximum of which each particular truck was capable under the test conditions. None of the trucks exceeded a speed of 30 miles an hour.

ACTIVITY IN HIGHWAY RESEARCH

Figures which show the interest of highway builders in research as a means of solving highway problems are given in a list of highway research projects recently compiled by the Highway Research Board.

According to the list there are under way at the present time 479 active projects, 205 of which are being conducted by the various State highway departments and the United States Bureau of Public Roads, 184 by universities, and 90 by municipalities, counties, and industrial organizations.

These researches are suggesting more economical methods of design and are pointing the way to better methods of construction, according to A. N. Johnson, chairman of the Highway Research Board.

A DEFORMATION TEST FOR ASPHALTIC MIXTURES

BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by **H. M. MILBURN**, Engineer of Tests

IN the design of asphaltic paving mixtures one of the first considerations is to produce a pavement which will retain its stability under high summer temperatures. Designers have been guided by past experience with the various classes of mixtures in producing a satisfactory pavement. There are no laboratory tests in general use for testing an actual specimen of the proposed mixture and it is felt that there is need for such a test.

The Bureau of Public Roads has developed a test, making use of new apparatus which determines the effect of varying the percentage of asphalt in a mixture by subjecting compressed specimens to a constant load at a constant temperature for a definite time and measuring the deformation.

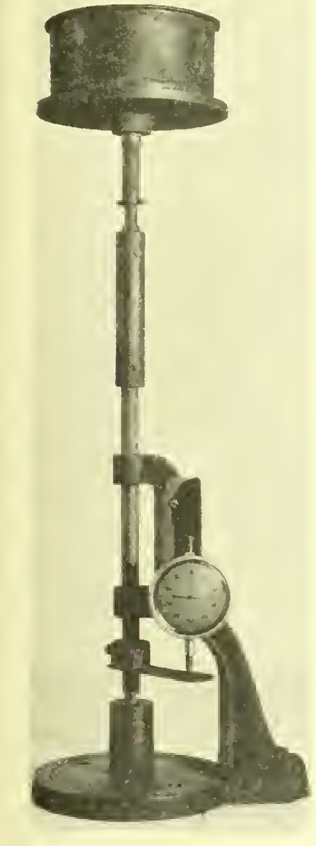

FIG. 1.—Apparatus used in making deformation tests

DESCRIPTION OF THE MACHINE

The deformation machine, shown in Figure 1, consists of the frame of a Vicat machine supporting a cylindrical rod in a vertical position. A cup for holding shot is attached to the upper end of the rod, the lower end being fitted into a cylindrical shoe 1 inch in height by 1¼ inches in diameter. Sufficient shot are placed in the cup so that the weight of the shot, cup, rod, and shoe is 5 kilograms. The decrease in height of the specimen is obtained by the use of an Ames dial reading to 0.001 of an inch attached to the frame, the foot of the Ames dial resting on a metal strip fastened to the rod of the machine. The size of the specimens tested is 1 inch in height by 1¼ inches in diameter. The machine is about 8 inches wide and 22 inches high. The deformation results given in this paper were obtained with this machine. Recently the machine has been redesigned in order to make it more compact, but in principle it is the same as the machine just described. This new machine, shown in Figure 2, differs from the former machine primarily in that two vertical parallel rods act as guides and the cup containing the shot is replaced by a rectangular weight. The machine is about 7½ inches wide and 12 inches high.

PREPARATION OF SPECIMENS

In the preparation of the specimens four different asphalts were used. Mixtures of each asphalt and a mineral aggregate consisting of 88 parts by weight of an asphalt sand and 12 parts by weight of a mineral filler were made. In three series of specimens the bitumen content was varied from 8 to 15 per cent by weight and in one series from 2 to 15 per cent, in each case the increments being 1 per cent. The mechanical analysis of the mineral aggregates used in the three series containing 8 to 15 per cent of bitumen and that used in the series containing 2 to 15 per cent of bitumen are given in columns (a) and (b), respectively, of the following table:

	(a)	(b)
	Per cent	*Per cent*
Passing a ¼-inch screen and retained on a No. 10 sieve	0.7	0.3
Passing a No. 10 sieve and retained on a No. 20 sieve	6.0	6.9
Passing a No. 20 sieve and retained on a No. 30 sieve	8.7	9.8
Passing a No. 30 sieve and retained on a No. 40 sieve	11.9	10.3
Passing a No. 40 sieve and retained on a No. 50 sieve	10.2	17.2
Passing a No. 50 sieve and retained on a No. 80 sieve	25.1	23.4
Passing a No. 80 sieve and retained on a No. 100 sieve	7.5	7.0
Passing a No. 100 sieve and retained on a No. 200 sieve	12.3	10.0
Passing a No. 200 sieve	17.5	15.1

In the preparation of the mixtures the asphalts and mineral aggregates were heated separately to a temperature of 300° to 325° F. and then thoroughly mixed in the proportions desired by hand, using a trowel or large spatula. In the making of the specimens a hollow steel mold 2½ inches in diameter and 3 inches in height was used. The diameter of the hole in the mold is 1¼ inches. The mold, which had been previously heated, was placed on a metal plate resting on the base of an Olsen testing machine and filled with an amount of the mixture sufficient to form a specimen substantially 1 inch in height. A cylindrical steel plunger 3½ inches long and $1\frac{15}{64}$ inches in diameter was then placed in the mold and pressure applied until the beam balanced at 2,000 pounds, using high speed. Then the balance weight was set at 10,000 pounds and the pressure applied at low speed until the beam just balanced. The pressure was then released and the specimen removed from the mold. The removal of the specimen was effected by placing the mold on a metal tube of sufficient height and diameter to allow free passage from the mold and forcing the specimen

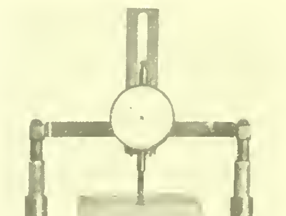

FIG. 2.—Improved design of apparatus to be used in future tests

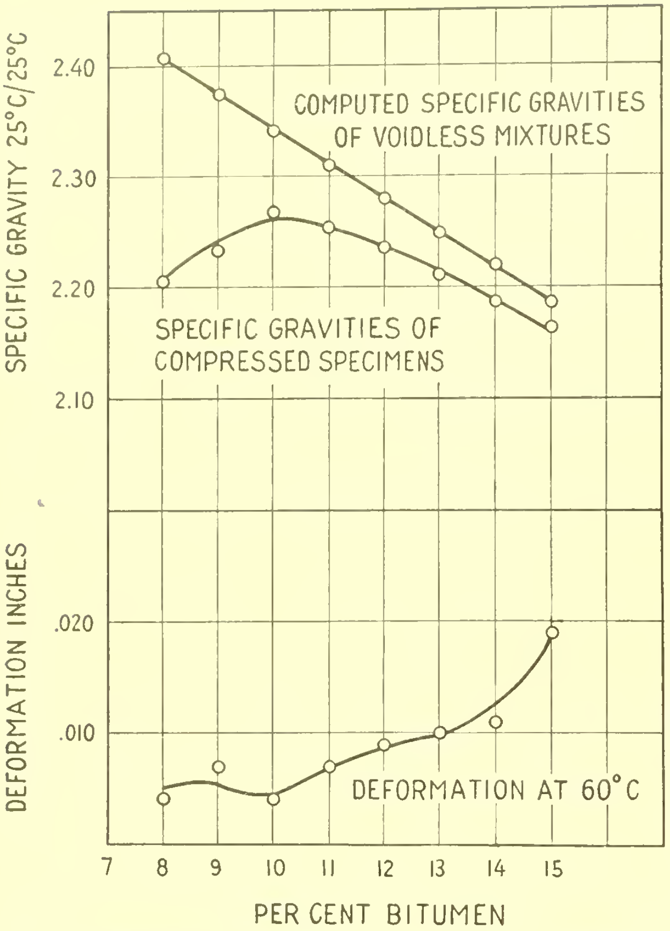

Fig. 3.—Results of series of tests on mixtures with asphalt L, having a penetration of 53

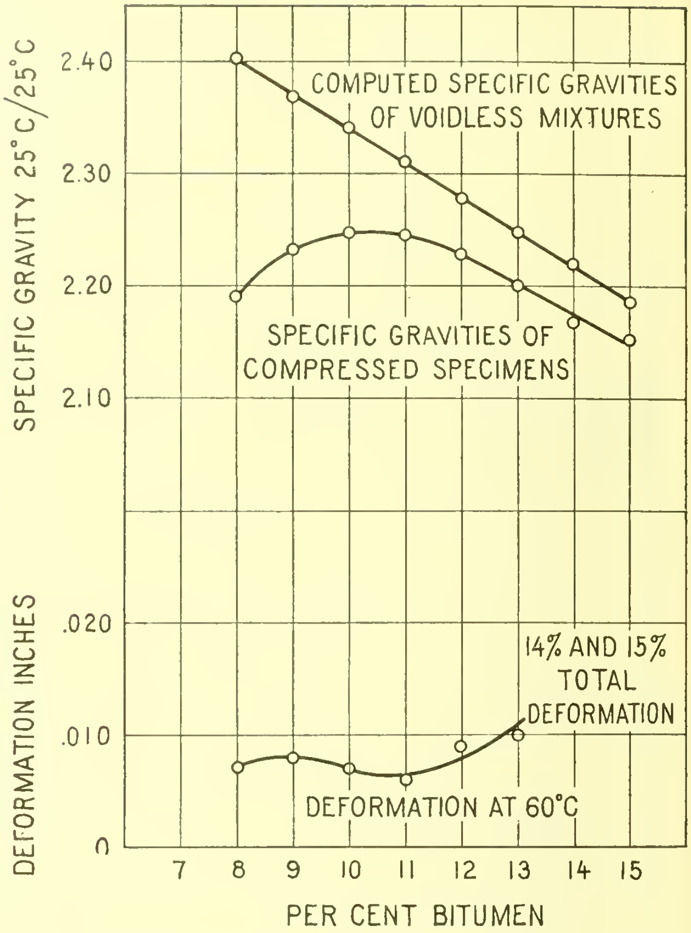

Fig. 4.—Results of series of tests on mixtures with asphalt J, having a penetration of 53

out by applying pressure to the plunger. The specific gravities of the specimens were determined by the displacement method.

While in these four series a pressure of 10,000 pounds was used exclusively, the pressure that was originally tried was 20,000 pounds as this pressure had previously been used with another mineral aggregate and specimens whose specific gravities seemed reasonable were obtained. (Results embodied in Progress Report of a Subcommittee of the Committee on Tests, A. A. S. H. O. for 1921.) In the present series, however, when the 20,000-pound pressure was tried, specific gravities which appeared abnormally high were obtained with specimens of low bitumen content. Furthermore, the specific gravities in general decreased from a maximum with 8 per cent of bitumen to a minimum with 15 per cent of bitumen, and analyses of the specimens with 8 per cent bitumen showed the mineral aggregate to contain 27.9 per cent of material passing the 200-mesh sieve, whereas the original mineral aggregate contained only 17.5 per cent. This increase in very fine material indicates that with this pressure of 20,000 pounds crushing of the mineral aggregate had occurred. It has been a matter of record[1] for many years that the voids in a sand decrease continuously to a certain point with the increase of mineral filler. It can be assumed, therefore, that the increase in fine material in the specimen had decreased the voids, resulting in high specific gravities.

TESTING SPECIMENS FOR DEFORMATION

The deformation machine was placed in an oven and heat so applied that a temperature of 60° C. was registered by a thermometer embedded in a specimen on the base of the machine. The specimen to be tested was kept in the oven for one hour. The load was then applied and the deformation of the specimen was obtained by noting the original reading of the Ames dial and the reading at the end of two hours, the apparatus remaining in the oven at constant temperature. In general, two specimens from each mix were tested. The deformations of specimens which collapsed during the test are designated as total and no attempt was made to measure the deformation of such specimens.

DISCUSSION OF RESULTS

Figures 3, 4, 5, and 6 give the results of the four series of deformation tests with corresponding specific gravities of the specimens and the calculated specific gravity on the assumption of a voidless mixture. The difference between these two specific gravities indicates the amount of voids in the specimen. In Figure 3 the deformations decrease from that of the specimen containing the 15 per cent of bitumen to and including that of the specimen containing 10 per cent of bitumen; then an increase occurs for the specimen containing 9 per cent of bitumen with a subsequent decrease for the 8 per cent specimen. Figure 4 shows that the deformation decreases from the 15 and 14 per cent

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[1] The Modern Asphalt Pavement, by Clifford Richardson, 2d edition, p. 82.

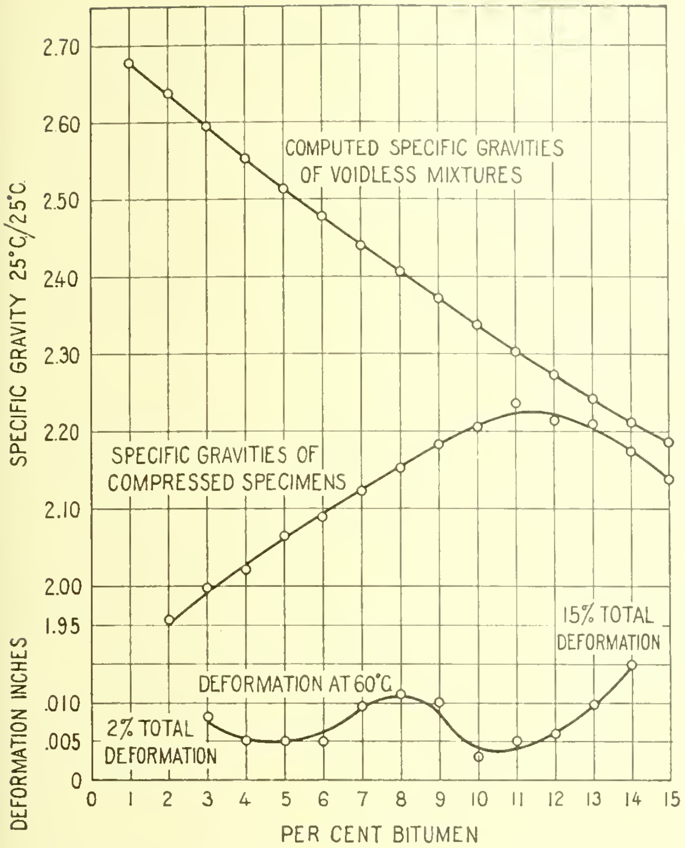

Fig. 5.—Results of series of tests on mixtures with asphalt 1, having a penetration of 36

specimens to the 11 per cent one, inclusive, and then increases up to and including the 9 per cent specimen, with a subsequent decrease for the 8 per cent specimen. Figure 5 shows that the deformation decreases from the 15 per cent specimen to and including the 10 per cent specimen. An increase in deformation then occurs up to and including the 8 per cent specimen, a decrease then occurring with subsequent increase resulting in total deformation with the 2 per cent specimen. These results appear to indicate that there are two fields of comparatively low deformations, one field embracing specimens of high specific gravities and the other field embracing specimens of very low specific gravities. In a correlation of specific gravity determinations with deformation determinations for the purpose of ascertaining the proper amount of bitumen to use with a given mineral aggregate, the field of deformations embraced in the field of high specific gravities should be the mixtures to be given greatest consideration.

The deformations shown in Figure 6 decrease from a maximum for the 15 per cent specimen down to and including the specimen of lowest bitumen content. Consequently, to determine the best percentage of bitumen to use it would be necessary to correlate the results of the deformation tests with other physical tests, and it is believed that specific gravity results should be given considerable weight.

The specific gravities given in Figures 3, 4, 5, and 6 all show the same general characteristics, i. e., they all increase in general up to a maximum and then decrease. As no voidless specimens were obtained, the specific gravities in no instance are as great as the calculated specific gravities of the voidless mixtures. It is of interest to note that in no series does the maximum specific gravity represent a specimen containing the minimum percentage of voids, the specimen of lowest voids in each series being a specimen with lower specific gravity than the maximum.

CONCLUSIONS

Results seem to indicate:

(*a*) That the effect at high temperatures of varying the proportions of an asphalt and a mineral aggregate of the type used in the wearing course of a sheet asphalt pavement can be ascertained by subjecting compressed specimens of the mixture to a constant static load at a constant temperature for a definite time.

(*b*) That by a correlation of the specific gravity results of the compressed specimens with the deformation results data are obtained relative to the proper proportions to use of a given asphalt and a given mineral aggregate in the design of asphalt mixtures of the type used in the wearing course of a sheet asphalt pavement. It is realized, however, that a deformation test at a high temperature may not necessarily indicate the proper proportioning of the mixture in order that it may best withstand low temperatures. Under such conditions it seems necessary to develop physical tests for mixtures at a low temperature for correlation with specific gravity and deformation results.

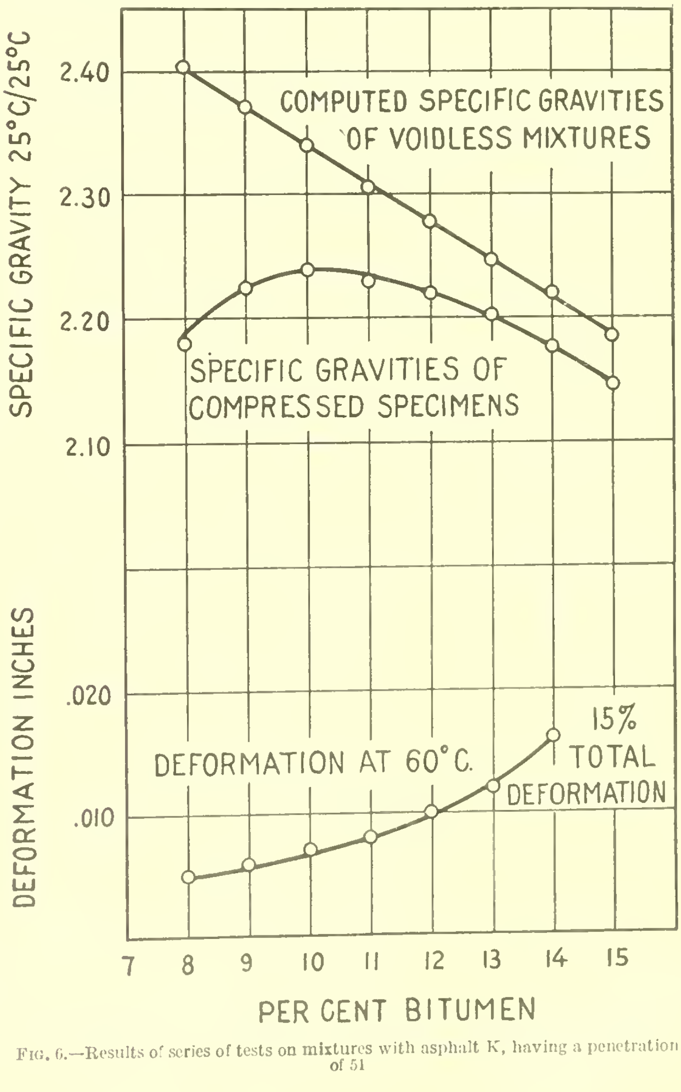

Fig. 6.—Results of series of tests on mixtures with asphalt K, having a penetration of 51

COLORS AND FORMS OF TRAFFIC SIGNALS

CODE PROPOSED BY SECTIONAL COMMITTEE OF THE AMERICAN ENGINEERING STANDARDS COMMITTEE

THE following is the text of a revised draft of the code of colors and forms for traffic signals for highways and vehicles submitted for approval to members of the sectional committee on code for traffic signals of the American Engineering Standards Committee.

The draft, which will probably be approved by the sectional committee, is as follows:

REVISED DRAFT OF CODE OF COLORS AND FORMS FOR TRAFFIC SIGNALS FOR HIGHWAYS AND VEHICLES

SECTION 1. PURPOSE AND SCOPE

Rule 10. Purpose.

The purpose of this code is to provide a standard of colors and methods, as applied to highway traffic and vehicles, which will promote uniform usage and thus decrease the likelihood of accidents, and conserve human life and limb. The approval of specific signs, glasses, or pigments as complying with the specifications herein contained should be based upon tests made by competent and disinterested organizations having proper facilities and recognized standing, with final appeal to the United States Bureau of Standards.

NOTE.—To secure the uniform application of this code, enforcing officers are urged to consult the committee which formulated it (or the American Engineering Standards Committee, 29 West Thirty-ninth Street, New York City) regarding matters of interpretation or items of dispute.

Rule 11. Scope.

This code is intended to cover the use of luminous and nonluminous signs and signals in connection with highway traffic, including moving and flashing signals, the use of lights, semaphores, and other signaling devices on vehicles.

SECTION 2. VEHICLE LIGHTS AND SIGNALS

Rule 20. Headlights.

(*a*) Headlights shall be white, amber, yellow, or any intermediate hue. If electric headlights are used, they shall conform to the rules governing the approval of electric headlighting devices for motor vehicles, of the Illuminating Engineering Society (tentative standard of the A. E. S. C., D2).

(b) No red or green lights shall be displayed upon any vehicle so as to be visible from a point directly in front of it.

Rule 21. Tail lights.

Tail lights shall be red, as seen from the rear.

NOTE.—Red has been adopted on account of its present universal use for this purpose. This use is not considered consistent with the general standardization of colors, and if a change is made in any jurisdiction, yellow is recommended.

Rule 22. Warning signal lights.

(*a*) Light signals displayed on a vehicle to indicate the driver's intention to reduce speed or change direction should be yellow. They shall not be red or green.

(*b*) Nonluminous mechanical signals displayed to indicate the driver's intention to reduce speed or change direction shall consist of a semaphore which can be projected from the left side of the vehicle. It is recommended that the position of the semaphore when projected be limited to the horizontal.

Rule 23. Marker lights.

Marker lights to indicate the dimension limits of bodies or loads should be located as near the upper left front corner as possible and should be yellow and visible from the front and rear.

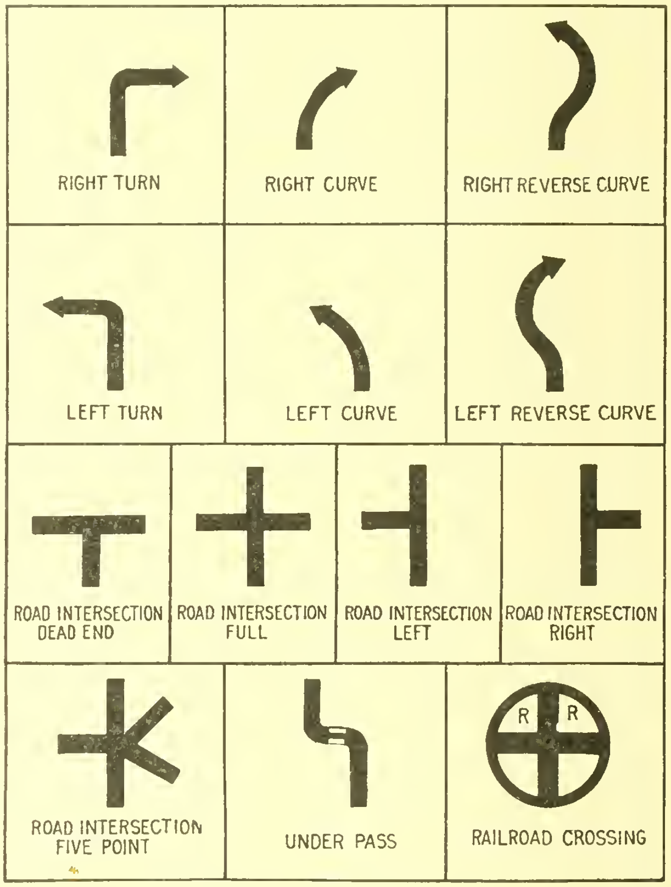

FIG. 1.—Symbols recommended by the sectional committee of the American Engineering Standards Committee

SECTION 3. HIGHWAY TRAFFIC SIGNALS—GENERAL

Rule 30. Significance of colors.

(*a*) Red shall be used as an indication to stop and for other purposes where required by law.

NOTE.—The use of red is proper as an indication to stop and to then proceed if conditions are favorable, as, for example, when "stop" and "proceed" regulations are in effect

(*b*) Yellow shall be used as an indication to exercise caution.

NOTE.—Yellow is appropriate when caution is to be exercised without stopping, as for partial street obstruction, so as to reserve red for a stop signal.

(*c*) Green shall be used as an indication to proceed.

Rule 31. Significance of form.

Where definite forms of luminous or nonluminous signs or signals are used either alone or in combination with colors to give the indications specified in Rule 30, the following will apply:

(*a*) Stop shall be indicated by having the greatest dimension horizontal, as with a horizontal semaphore or string of lights.

Note.—Where the word STOP is used to direct traffic, it should be in red letters arranged horizontally.

(*b*) Caution shall be indicated by having the greatest dimension at an angle of 45° with the horizontal when an inclined semaphore or string of lights is used.

Note.—Cautionary signs complying with Rule 33 may be used.

(*c*) Proceed shall be indicated by having the greatest dimension vertical, as with a vertical semaphore or string of lights.

Note.—Where the word GO is used to direct traffic, it should be in green letters arranged vertically.

Rule 32. Significance of flashing.

Flashing luminous signals shall conform to the use of colors prescribed in Rule 30.

Rule 33. Cautionary signs for drivers.

(*a*) Highway signs and signals of a cautionary nature for the notice of drivers shall have letters and symbols black on a yellow background or yellow upon a black background.

Note.—It is recommended that all cautionary signs have a background of one distinctive shape, and that such shape be reserved for this purpose.

(*b*) For signs to be read from vehicles in motion symbols are recommended in place of, or supplementary to, words.

Note.—When both symbols and words are used, symbols should be given greater prominence.

(*c*) When symbols are used, those illustrated in Figure 1 are recommended for the situation indicated. Other or special symbols should be such as not to be confused with these.

Note.—The symbol for the five-point road intersection is to be varied to fit each specific case.

Rule 34. Location of signs.

(*a*) Traffic signs shall be erected with the top not more than 8 feet above the surface of the roadway.

(*b*) Traffic signs shall be erected at the outer edge of the shoulder area and on right-hand side of roadway where practicable.

Rule 35. Cautionary signs for pedestrians.

Signs addressed to pedestrians shall consist of blue letters upon a white background.

Section 4. Signals at Railroad Grade Crossings

Rule 40. Aspect.

An electrically or mechanically operated signal used for the protection of highway traffic at railroad crossings shall present toward the highway when indicating the approach of a train the appearance of a horizontally swinging red light and (or) disk.

Note.—This covers the use of so-called wigwags and of alternately flashing red lights, and the use of these devices should be restricted to the purpose of indicating the approach of a train.

Rule 41. Location.

The railroad standard highway crossing sign and the signal shall be mounted on the same post.

Rule 42. Operating time.

Automatic signal devices for indicating the approach of trains shall be so arranged as to indicate for not less than 20 seconds before the arrival at the crossing of the fastest train operated on the track. Local conditions such as three or more tracks, bad approaches, etc., should be allowed for by increasing the operating time, bearing in mind that too long an operation by slow trains is undesirable.

Rule 43. Flashing light type.

(*a*) *Height.*—The lamps should preferably be not less than 6 feet nor more than 9 feet above the surface of the highway.

(*b*) *Width.*—The two lamps shall be mounted horizontally 2 feet 6 inches centers.

(*c*) *Flashes.*—Lights shall flash alternately. The number of flashes of each light per minute shall be 30 minimum, 45 maximum.

(*d*) *Hoods.* Lamp units shall be properly hooded.

(*e*) *Range.*—When lamps are operated at normal voltage, the range, on tangent, shall be at least 300 feet on a clear day, with a bright sun at or near the zenith.

(*f*) *Spread.*—The beam spread shall be not less than 3° each side of the axial beam under normal conditions. This beam spread is interpreted to refer to the point at the angle mentioned where the intensity of the beam is 50 per cent of the axial beam under normal conditions.

(*g*) *Lenses or roundels.*—The size shall be 5⅜ inches minimum, 8⅜ inches maximum.

Rule 44. Wigwag type.

(*a*) Length of stroke is the length of chord which subtends the arc, determined by the center of the disk in its extreme positions, and shall be 2 feet 6 inches.

(*b*) *Disk.*—The disk shall be 20 inches in diameter. Its field shall be white with a black circumferential border 1 inch wide. The horizontal and vertical diameters shall be shown by black lines 2½ inches wide. A red lens or roundel should be placed at the center in front of the lamp.

(*c*) *Number of cycles.*—Movement from one extreme to the other and back constitutes a cycle. The number of cycles per minute shall be 30 minimum and 45 maximum.

(*d*) The lamp, with which the disk shall be equipped shall be lighted when the disk is swinging.

Rule 45. Approach signal.

Advance warning signals which indicate approach to a railroad crossing, and not the actual approach of a train, shall conform to the provisions of Section 3, Rule 30.

Section 5. Specifications for Colors

Rule 50. Definition of Colors.

Red, yellow, green, or blue as used in this code are intended to mean:

(*a*) The colors resulting from the transmission of the proper light through the proper glasses and having the characteristics described in Rules 51 and 52.

(*b*) The colors resulting from the reflection of white light from the proper pigments and having the characteristics described in Rule 53.

NOTE.—*For a definition of white light, reference is made to the report of the Colorimetry Committee of the Optical Society of America, 1920–21 (Journal of the Optical Society of America and Review Scientific Inst. 6, p. 563; 1922). Substantially, it is average sunlight at noon at latitude of Washington.*

Rule 51. Qualitative definition of colors for luminous signals.

(*a*) *Red.*—The spectrum of red shall contain both red and orange but not more than a trace of yellow and no green, blue, or violet. The most desirable hue is entirely free from yellow which means that the glass does not transmit the yellow light from a sodium flame.

(*b*) *Yellow.*—The spectrum of yellow shall contain red, yellow, and green, with but little blue and no violet. The most desirable hue is entirely free from blue and might be designated a light amber.

(*c*) *Green.*—The spectrum of green shall contain yellow, green, blue, and violet, with only a trace of red and orange. This hue is known as "admiralty green" and has a bluish tint when observed by daylight.

Rule 52. Quantitative definition of colors for luminous signals.

The colors red, yellow, and green shall have the following characteristics:

	Dominant wave length	Purity	Integral transmission of glass
	Millimicrons	*Per cent*	*Per cent*
Red	Not less than 624	Not less than 100	Not less than 10.
Yellow	Not less than 592 nor more than 600.	Not less than 97	Not less than 24.
Green	Not less than 496 nor more than 536.	Not less than 45	Not less than 11.

These values are determined by the transmission of light from a source at the color temperature of 2.360° K (practically that of the acetylene flame or present type of vacuum tungsten lamp at normal voltage) through the respective glasses. They are based upon spectral transmission measurements and upon computations carried out in accordance with the methods and data described in the Colorimetry Report of the Optical Society of America.

NOTE.—The light and dark limits of the glasses on which the above values are based have the following relative transmissions on the scale of the American Railway Association.

	Light limit	Dark limit
Red	300	150
Yellow	200	100
Green	250	100

Rule 53. Quantitative definition of colors for nonluminous signs.

The colors red, yellow, green, and blue shall have the following characteristics:

	Dominant wave length	Purity	Integral reflection of pigment
	Millimicrons	*Per cent*	*Per cent*
Red	Not less than 608	Not less than 60	Not less than 8.
Yellow	Not less than 580 nor more than 588.	Not less than 80	Not less than 35.
Green	Not less than 524 nor more than 552.	Not less than 30	Not less than 8.
Blue	Not less than 466 nor more than 474.	Not less than 10	Not less than 4.

These values are determined by the reflection of white light from the respective pigments. They are based upon spectral reflection measurements under conditions of diffuse illumination and upon computations carried out in accordance with the methods and data described in the Colorimetry Report of the Optical Society of America.

(Continued from page 123)

closer together than in the case of either the slump or flow test, indicating that determinations of consistency by this method are more independent of the type of aggregate than in the case of either of the other methods. In view of the fact that the compressive strength of the concrete appears also, in general, to be independent of the type of aggregate, this would appear to be an advantage. It will be noted also that the slope of the average curve for consistency as determined by the plate test appears to parallel in general the curve showing the relation between water-cement ratio and strength, It will be seen, for instance, that for the leaner mixes, such as 1:2½:5 and 1:3:6, the curves are much flatter than in the case of the richer mixes. The 1:2:3 mix shows the steepest slope and likewise the greatest range in consistency which is to be expected in view of the high plasticity or workability of a mix of this nature.

Observing the results of the flow and slump tests, we find that the points representing the various types of aggregates are much more widely separated. In the slump test, for instance, it will be observed that for a given water-cement ratio the points representing the tests with the gravel aggregate lie in general considerably higher than in the case of either the stone or slag concrete, while the stone in general is next and the slag lowest. These results are in accord with general experience with this test as a measure of consistency. The slump test appears likewise to be much more erratic for leaner mixes than for the richer mixes. This is also borne out by experience. The flow test results are fairly concordant for each type of aggregate. They, however, show the same general separation by groups as is indicated in the slump test.

For the purpose of checking the value of this device as a method of field control, a series of tests was run in connection with a concrete paving project. On this project rather poorly graded crushed limestone was being used as coarse aggregate, with a consequent tendency toward segregation whenever too much water was used. By the use of this device it was possible to control the mix so that a considerably drier consistency was obtained with consequent freedom from segregation, besides undoubted increase in strength.

The authors feel that enough work has been done with the plate device to warrant its presentation as a possible method of field control of the consistency of concrete. Extreme accuracy is not claimed. In no case is it possible to check closer than one or two pounds on the plate. However, the data which have been obtained indicate that this is sufficiently accurate for all practical purposes. The fact that the test gives practically the same results irrespective of the type of aggregate would appear to be a point in its favor, as would also the fact that, for a given mix, it appears to be a direct measure of the crushing strength of the concrete.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583. Report on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916. 10c.
*660. Highway Cost Keeping. 10c.
*670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691. Typical Specifications for Bituminous Road Materials. 10c.
*704. Typical Specifications for Nonbituminous Road Materials. 5c.
*724. Drainage Methods and Foundations for County Roads. 20c.
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
No. 1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1259. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULAR

No. 94. TNT as a Blasting Explosive.

FARMERS' BULLETIN

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*505. Benefits of Improved Roads. 5c.

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*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.

OFFICE OF PUBLIC ROADS BULLETIN

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59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
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161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JULY 31, 1925

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863,197.85	611.8	$ 32,469.80	$ 12,642.50	0.8	$ 15.417,487.54	$ 7.482.213.97	804.0	$ 300,107.34	$ 146.749.34	24.8	$ 2,290,029.33	Alabama
Arizona	9.580,133.43	6,015.119.94	513.8	237.925.80	145,397.06	7.1	2,049,599.63	1,270,492.41	157.3	176.977.10	108.150.70	25.7	2,011.711.89	Arizona
Arkansas	13,310,190.08	5.380.181.73	1.048.9				7,175,728.71	3.252.153.53	387.4	862.255.62	398.228.65	71.8	1,295,990.09	Arkansas
California	22,345,175.99	10,719.249.61	894.8				10,605,687.35	5,043.509.43	253.5	727,950.93	360.660.72	41.5	3,442,522.24	California
Colorado	11.875.703.94	6,067.814.34	651.2	47.462.25	25,635.82	4.0	4,072,242.93	2.043,234.77	184.8	999.517.75	503,000.34	54.8	2,292,432.73	Colorado
Connecticut	4,558,639.29	1.819.358.65	101.6	305,300.83	95,200.00	4.8	1,796,870.72	550,037.75	28.1	371,355.48	137,522.73	5.5	1,252,865.86	Connecticut
Delaware	4.281.659.81	1,495,190.65	107.1				489,503.74	213.581.10	12.3	1,023.384.85	388.799.75	22.6	6.583.50	Delaware
Florida	2.959.273.72	1.405.487.97	96.3				8,787,223.95	4,279.162.51	251.8	115.547.09	91.547.09	0.1	1.403,567.43	Florida
Georgia	20.156,002.37	9,406.366.45	1.478.3	187,801.21	92,983.55	5.3	11.181.335.94	5.515,705.40	747.2	985,295.04	443,523.45	57.8	974.406.13	Georgia
Idaho	9,394,675.80	4.815.332.25	600.1	26.406.06	14.540.60	1.0	2,411.772.54	1.512,125.19	151.4	501,215.03	311.867.09	40.9	960.773.86	Idaho
Illinois	40.010,481.10	18,640.075.28	1,235.2				8.350,804.09	4,041.970.89	281.2	487.835.78	243,917.89	17.4	3,702,005.94	Illinois
Indiana	13.539.172.85	5.562.455.68	422.1	227,828.34	104.962.95	7.5	15,592,088.38	7,616,237.24	465.5	963,429.82	467.580.58	25.6	1.500,748.54	Indiana
Iowa	27.272.285.21	11,107,492.99	1.995.9	102,280.50	39,496.95	15.4	7,425.789.49	3.275.441.22	418.3	978,898.49	477,595.00	99.7	2,505.507.84	Iowa
Kansas	26.399.695.77	9,756,273.32	831.4	769.544.49	299.522.80	17.5	11.987.431.14	5,098.719.21	582.7	996.527.64	388.438.30	54.5	1.831.695.37	Kansas
Kentucky	14.832,324.28	6,205.994.69	584.9				8,298.279.65	3.724.887.09	299.0	690.422.33	295,211.15	40.1	1.657.253.17	Kentucky
Louisiana	11.939.424.97	5.279.870.85	927.5	746.114.83	349.217.80	54.8	2.814.895.62	1.346,385.48	115.2	356,130.73	178.055.35	2.8	1,109.163.50	Louisiana
Maine	8,174.281.31	3.907,870.33	281.4	173,275.16	85.733.59	5.6	1.311,336.31	561.480.38	42.2				1.230.027.70	Maine
Maryland	8,132.505.90	3.849,383.16	294.4				2,070.460.55	884.007.47	56.7	1,139.721.78	546.339.25	76.2	5.003.12	Maryland
Massachusetts	14,047,565.22	6,467.661.28	300.8	198,725.73	53.820.00	2.7	6,378.944.39	1.411,819.74	82.5	715.821.54	225.335.00	13.4	1,850.261.98	Massachusetts
Michigan	15.234.000.80	7.328.315.91	512.6	894.484.74	419.344.47	50.9	14,227.345,89	5.655,371.05	432.4	594.848,44	276,168.19	0.6	3.424.798.38	Michigan
Minnesota	30.415.685.89	12.738.642.04	2,721.2				7,923.339.62	3.696.800.00	739.3	2,227.520.82	608.400.00	99.3	498,727.96	Minnesota
Mississippi	10.292.285.79	4,988.702.73	803.4	134.986.30	67.493.15	12.9	8,475.317.22	4.233.402.97	469.7	1.132.183.38	666,051.56	59.9	967.572.50	Mississippi
Missouri	17,368.156.57	8.219.411.43	1,118.9	533.614,80	316,927.78	43.9	25.404.724.60	11,037.054.68	825.3	528.298.05	202.679.59	13.4	582.931.42	Missouri
Montana	10,165.600.41	5.317.523.15	921.5	123.476.31	56,491.30	9.5	1.976.050.87	1.477.525.00	173.3	988.759.09	551,613.73	117.1	5,102,735.82	Montana
Nebraska	9.305.374.36	4.389.523.50	1.670.6	511,445.26	256,485.50	29.0	8.066,040.35	3,886.052.25	820.1	1.673,775.11	786.056.55	87.0	3.715,788.19	Nebraska
Nevada	4.917.455.69	3,088,299.78	367.3	88.333.61	73,261.46	8.0	6.185.772,57	4.346.943.29	442.6	80.728.23	70.814.79	15.9	260.087.68	Nevada
New Hampshire	4.166,687.85	1.985.225.87	208.1				1.177.495.51	566.610.59	37.7	30,791.75	12,495.00	0.8	245.355.54	New Hampshire
New Jersey	11,951,357.45	3,820.579.99	219.1	183.975.37	48,630.00	3.2	9.289.268.95	3.311.777.62	57.3	1,346.330.48	304.408.53	21.2	38,832.85	New Jersey
New Mexico	8.717.999.18	4,914.070.51	1,081.3	491.955.02	413.572.15	29.6	3.834.381.19	2,441,722.52	355.1	383.618.39	239.309.08	29.1	1.766,723.64	New Mexico
New York	28,597.769.67	12.229.076.53	831.5	2,380.670.38	934.192.29	59.7	28,262,125.75	9.454.545.10	578.0	9,832,600.00	2.593,351.46	178.6	6.054.078.63	New York
North Carolina	21,014.450.41	8,745.454.69	1.119.8	483.841.87	173.882.00	18.1	7,658.702.09	3.323,201.00	184.5	2,245.098.85	978.672.58	68.5	771.208.83	North Carolina
North Dakota	10,829,263.82	5.268.930.47	1,917.5	131.549.77	55,824.88	40.4	3.505,541.81	1.768.931.82	471.2	934.955,95	457.483.37	131.0	1.973.184.45	North Dakota
Ohio	41.672.252.81	16.244.993.93	1.191.1	812.253.81	238.966.05	19.3	8.968.966.26	3,305.335.70	275.8	5.461.257.51	1,955.145.65	140.8	2,183.310.67	Ohio
Oklahoma	20.787.024.94	9,672,890.34	852.2	772.371.50	329.961.36	28.6	5.253.876.77	2,983,035.94	283.4	1.305,037.25	518,952.08	69.7	685.978.29	Oklahoma
Oregon	14,388.188.70	7.142.354.63	794.5	169,042.13	90.512.52	9.8	2.887.797.48	1.555.475.29	127.5	774.854,82	455,603.56	59.8	321.770.01	Oregon
Pennsylvania	43,054.835.19	16,222,023.97	850.3				32.354.835.50	8.695.355.14	660.3	7.213,528.55	2.152,295.42	162.4	992.054.47	Pennsylvania
Rhode Island	2.628,495.20	1.119.688.09	64.8				2.372,855.09	533.939.59	34.0	87,534.25	20.100.00	1.3	524,938.32	Rhode Island
South Carolina	11.153.347.84	6,121.267.54	1.235.9	389.582.68	172,605.08	18.1	5.223.547.25	2.798.033.10	355.1	870.524.91	354.687.36	36.1	293,501.93	South Carolina
South Dakota	12.091.434 57	5.989.878.00	1.447.9	507.689.25	235,395.17	54.7	7,349,819.54	3,604,638.90	990.6	295.179.39	77,261.73	56.0	25,524.20	South Dakota
Tennessee	13.789.140.98	8.732.079.77	497.9	1,614,333.34	792,240.45	48.6	10.501,567.75	4.722,108.22	374.1	128.336.55	64.168.27	10.8	1.337.025.29	Tennessee
Texas	54.120.970.83	21,057,940.12	3.907.1	965.510.88	386.699.47	82.8	24.643.272.32	10.178.618.25	1.433.1	2.574,527.05	1,075.262.80	213.3	3,441.507.35	Texas
Utah	5,259,159.41	3.818,835.91	423.1	65,046.59	40.114.35	5.8	2.948.945.47	2,001,375.22	220.3	367,448.27	270,355.59	36.5	832.257.82	Utah
Vermont	3.015.174.51	1.452,894.46	107.8				1.219.636.45	684.290.42	25.5	488.824.92	164,815.73	10.3	.597.603.40	Vermont
Virginia	13.099.720.01	5,271.998.20	675.2	845.755.70	422,543.92	43.3	9.474.535.35	4,201.128.43	309.7	1,958.811.45	881.787.43	70.1	256.208.02	Virginia
Washington	13.352.504.18	5.117,211.87	525.7	1,049.945.31	437.338.54	25.0	2.094.224.65	988.900.00	102.3	833.280.25	351,000.00	22.4	1,101,214.49	Washington
West Virginia	7.343,200.85	3.230,293.33	325.7	212.859.07	103.111.28	11.5	4.985,938.10	2,074,055.28	137.2	2,055.068.51	754.610.03	49.9	389,447.08	West Virginia
Wisconsin	21.807.140.91	8,919,540.62	1.451.7				4.614.450.56	2,211.353.25	213.8	1.208,300.43	595,037.00	74.3	3.824,728.13	Wisconsin
Wyoming	8.809.819.33	4,739.095.57	982.0	198.443.81	125,165.00	18.8	3.724.227.32	2.334.689.63	254.2	480.564.59	309,066.00	38.0	113,281.80	Wyoming
Hawaii							342,277.22	97,440.00	5.5				533,910.00	Hawaii
TOTALS	740,140,790.82	325,654,346.00	41,898.3	16,708,314.59	7,531,091.91	800.1	384,062,463.52	168,181,717.94	16,644.2	60,498,115.62	23,593,103.74	2,559.2	73,289,740.41	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 99.453.213.06 Federal aid $ 45.298.750.40 Miles 4.367.9

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 6, NO. 7 ▽ SEPTEMBER, 1925

IMPROPER DESIGN PERMITS SURFACE WATER TO SATURATE AND SOFTEN SUBGRADE SOIL UNDER EDGE OF PAVEMENT

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THE PRESENT STATUS OF SUBGRADE STUDIES

Reported by A. C. ROSE, Associate Highway Engineer, U. S. Bureau of Public Roads

The review of the progress of subgrade research to which this entire issue is devoted is the first of a series of similar articles which will present an abstract of the results of highway research in all its phases. Other articles dealing with the status of research affecting the design and construction of road surfaces, the economics of highway transportation, highway finance and administration, and other aspects of the highway situation will appear in later issues.

Glenn Frank, the editor of the Century Magazine, who retires this month to assume the presidency of the University of Wisconsin, writing in his valedictory editorial of the need of utilizing the contributions of research in all the sciences as the basis for a renaissance of western civilization, has so fully expressed our idea in presenting these reviews that we can no better explain our purpose than by quoting a single paragraph.

"The end of all research and analysis," says Mr. Frank, "is synthesis and social application. This must mean, it seems to me, that every now and then we must gather up the results of a period of research into what, for want of a better term, may be called a series of tentative dogmatisms upon which society can act until further research reveals wiser bases of action."

Mr. Frank is referring, of course, to a broad application of the product of all the sciences to the condition of society in general, but if he had been writing directly to highway engineers he could not have touched more accurately upon the present need.

A brief exposition of the existing knowledge of highway subgrades should be of value at this time in order to bring together in one paper the principal findings. From these the trend of the investigations may be indicated, the missing data may be identified, and the specific studies may be determined which are necessary in order to make the next step in the progressive development of this subject. In general, this article will discuss the results of experience and research which add to the possibility of distinguishing between good and bad subgrade soils. By quotations from test data gathered from many sources it will present the concrete findings which may now be practically applied to determine the design of a pavement on a specific kind of subgrade; and, by revealing the missing links in the chain of information, it will indicate the further research immediately required to accelerate the progress of our knowledge concerning this fundamental phase of highway design and construction.

PRINCIPAL FINDINGS OF SUBGRADE STUDIES TO DATE

THE QUANTITY AND CHARACTER of the clay content of a soil in the United States seems in general to determine whether it will make a good or bad subgrade.

Laboratory and field tests have been devised for distinguishing between good and bad subgrade soils.

The moisture equivalent percentage seems to be a critical value in respect to the bearing power of a subgrade soil. When wetted beyond this percentage, the bearing power seems to fall off rapidly.

At a depth below the surface of the soil sufficient to be removed from the influence of surface water and other forms of free water, there are indications that the moisture content rarely exceeds the moisture equivalent percentage.

There are indications that by proper subgrade design it may be possible to control the moisture content of a subgrade soil to a maximum value approximately equal to the moisture equivalent percentage.

Construction methods which may be used to overcome the effect of bad subgrade soils are as follows:

(*a*) Use coarse-grained soils for building fills over heavy clay soils.
(*b*) Use side ditches of special design.
(*c*) Use tile drains beside but not under the pavement.
(*d*) Use a granular subbase.
(*e*) Thicken the pavement.
(*f*) Add steel reinforcement.

It is gratifying to note the rapid advance toward the solution of the subgrade problem which has occurred during the past decade. Prior to this period one might have searched through all the engineering literature and found nothing more than general and indefinite observations on the subject. This discussion will contrast these general observations with the more scientific treatment to which the problem has recently been subjected, prominent in which have been the laboratory and field tests of the Bureau of Public Roads, the Pittsburg test road in California, and the Bates test road in Illinois, together with the contemporaneous studies of the various State highway departments.

Reference will be made principally to the specific data which are believed to be of practical use in highway design. Theoretical studies which up to this time do not seem to have developed any principles of practical value will be avoided.

An illustration of this is the difference of opinion which seems to exist concerning the character and effect of the colloidal fraction of a soil. One of the latest concepts of this matter seems to have been given by P. L. Gile, of the United States Bureau of Soils.[1] The following quotations from this article are believed pertinent:

"It is now recognized that the colloidal material is not of definite composition and that it may form a large part of the whole soil. * * *

"The quantity of colloidal material in the soil probably varies from a trace to almost 100 per cent. Usually, loams contain from 20 to 25 per cent of colloid; coarser textured soils contain less; and clays usually contain from 40 to 50 per cent, although they may run much higher. A few soils that have been examined appear to be more than 90 per cent colloidal material. These figures are not based only on determinations by ratio methods. From a sample of Houston clay soil 65 per cent of colloidal material was actually isolated. By far the greater part of the isolated material was made up of submicroscopic particles. * * *

"The quantity of colloidal material in a soil does not appear to differ greatly, as a rule, from the quantity of the old 'clay fraction' given by various systems of mechanical analysis. It is now evident, however, that, in certain cases, the nature of the clay fraction (which is largely colloidal material) may be a more important factor in determining how a soil will act than the quantity of this separate. * * *."

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[1] Recent investigations of soil colloids, by Philip L. Gile. Proceedings of the American Society of Civil Engineers, vol. 51, No. 5, May 1925.

It is evident from these quotations that the free colloidal material which may be isolated from a soil sample by the process of centrifuging may represent only a portion of the entire colloidal material present or else all of it. The balance of the colloidal material seems to be in a combined form and exists in particles larger than those which may be identified by centrifuging.

The above data, therefore, confirms the conclusions arrived at by the Bureau of Public Roads that the portion of the suspension clay in a soil which may be isolated by centrifuging is usually such a small fraction of the entire soil sample that its effect upon the various physical properties of the soil is of minor importance as compared with the effect of the entire clay fraction. An examination of the laboratory determinations of the bureau shows that the variation of the volumetric shrinkage values and other characteristics of 57 samples of different soils do not vary with a corresponding variation in the suspension clay (which resembles the colloidal material) but rather with the variation of the entire clay fraction including the suspension clay. [2]

After giving examples which indicate the trend of engineering opinion, an attempt will be made to harmonize the data and by synthetic process to bring out general conclusions which are representative of the most advanced practice. These will include construction methods used on bad subgrade soils. It will be evident that our present knowledge is far from satisfactory either as to degree or extent. The paper will conclude, therefore, with brief allusions to the studies which are believed of critical importance at this time in order to hasten the solution of the problem.

QUANTITY AND CHARACTER OF THE CLAY CONTENT SEEM TO DETERMINE SUITABILITY OF SUBGRADE SOIL

All the data submitted in this discussion point to the conclusion that, in the United States at least, the amount and character of clay in a soil determine its suitability for a highway subgrade. The results enumerated here should not be applied in foreign countries without a careful check against the local conditions. As Mr. H. H. Bennett [3] points out:

"In humid tropic countries there are very extensive areas of clay of exceedingly fine texture, which are markedly unlike the common clays of the temperate zone in their physical characteristics—notably in their open nature. The colloidal material has reached an advanced stage of weathering which gives the property of flocculency or a tendency to segregate into clusters, and the colloids do not mix with water readily. This gives rise to a condition of porosity which permits ready downward passage of rainwater. In parts of Central America there are clays of this nature that will plow into a good condition of tilth immediately after a rain or even while rain is falling. Types of this kind possess but slight plasticity and stickiness, and will serve admirably as subbase material even where almost the entire mass is made up of particles as fine as the finest clay. In Cuba large areas of very fine-textured red land are so porous that practically all of the rain passes downward, the run-off being insufficient to develop streams or drainage depressions. In the humid tropics as in the humid temperate zone, the pale yellow, bluish and mottled clays will be found least satisfactory for subbase material.

"In determining the clay content of soils with an exceptionally high percentage of organic matter, some deduction should be made for the organic matter, since this material, if thoroughly decomposed will be largely colloidal and will appear in the results as clay. In most clays this feature may be disregarded as the humus will probably be in that form which will not materially lessen the plasticity."

Generally speaking, however, all the data submitted in this country indicate—the character of the clay remaining constant—that good subgrade soils have a low clay content by mechanical analysis and bad subgrade soils are characterized by a high percentage of clay.[4] Some exceptions to this general statement have been pointed out by Mr. H. H. Bennett.

"Certain materials occurring in both the humid and arid regions, but much more commonly in the latter, will show in the mechanical analysis a composition as fine grained as the heaviest clay, and yet have very different physical characteristics from normal clays. These materials have a common property of being chalky. They are composed largely of carbonate of lime, or sulphate of lime, or both, and are known by such names as 'caliche,' 'plains marl,' 'gyp beds,' and 'rotten limestone.' All of them are low in content of colloids, at least in that type of colloids which impart plasticity and water-logging properties common to plastic clays.

"In the presence of excessive moisture the tendency of these materials is to remain firm rather than to become soft and miry. For this reason they can be relied upon generally as constituting good subbase soil for roads. The 'rotten limestone' formation is of very extensive occurrence in the Black Waxy Belt of Texas and southeastern Oklahoma, and the prairie region of Alabama and Mississippi; while the other materials underlie or outcrop over vast stretches of the Great Plains and arid regions.

"Clays of whitish, bluish gray, and pale yellow colors, or those showing a mottling or streaking of these colors, along with red, usually will be found more unfavorable (more subject to water-logging and lacking in firmness) than clays of red, brown, and uniform deep-yellow color. The common red clay of the eastern Piedmont region, for example, should make a good subbase for highways because it is of a more open nature and allows excess water to pass through."

Further studies may bring out the adverse qualities of the clay soils which will predetermine their usefulness for subgrades. At the present time, however, it seems that an increase in the quantity of the clay fraction, its character remaining constant, will increase the amount of heaving when the soil is frozen, increase the volume change of the soil due to shrinkage and swelling with variations in the moisture content, make the soil less permeable to water and thus make tile drains ineffectual in most cases in heavy clays, lend an elastic property to the soil when subjected to reversals of load, make it liable to permanent deformation when subjected to repeated or excessive loads when moist, increase the moisture-holding capacity of the soil, make the soil plastic when wet, increase the amount but decrease the rate of capillary movement of water, and reduce the allowable maximum bearing value of the soil.

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[2] Physical properties of subgrade materials; by J. R. Boyd, Table 1. Proceedings of the American Society for Testing Materials, vol. 22, 1922, pt. 2, p. 350.

[3] Soil Scientist, United States Bureau of Soils.

[4] Researches on the structural design of highways by the United States Bureau of Public Roads, by A. T. Goldbeck. Transactions of the American Society of Civil Engineers, vol. 88, p. 264, 1925.

DISCUSSION INCLUDES OUTSTANDING OBSERVATIONS OF LAST DECADE

Some of the extraordinary observations concerning the behavior of subgrade soils which have been made by various investigators during the past decade are enumerated in this paper with the hope that they will be useful in solving the problems of some particular locality. These observations are given in detail at random without any attempt to correlate the individual cases because of the wide differences in degree and kind. They will bring out, for example, the moisture-resisting quality of the dry, brown silt loam of the Bates Road tests; the effects of admixtures such as granular material, Portland cement, and hydrated lime; the merits of a sand cushion under macadam roads; the use of tar paper on the loess soil of western Iowa to prevent hair cracking in concrete pavements during the curing period; the great volume changes occurring in adobe-soil subgrades; the value of layers or subbases of sand, gravel, or other granular material; the worst pavement failures prevalent in clay cuts of excavations; the impracticability of the selection and substitution of subgrade materials as a general solution of the subgrade problem; the adequacy of topsoil and sand-clay methods where local conditions permit but the restricted use of this process as a general solution of the subgrade problem;[5] and the enumeration of the laboratory and field tests which are now in use.

SUBGRADE SOIL STUDIES SEEN AS URGENTLY NEEDED

A CLASSIFICATION OF SOILS to conform as nearly as possible to the nomenclature and grading used by the United States Bureau of Soils in its soil survey bulletins.

The adoption of standard methods for making laboratory and field tests on subgrade soils.

The determination of the distribution of the maximum pressure through pavements and subbases of various types and thicknesses to subgrades of various soil types.

The determination of the maximum intensity of pressures permissible on soils of various types such as sand, silt, and clay. These maximum pressures for the same soil may differ in proportion to the area in contact with the subgrade resulting from the character and possibly the thickness of the pavement.

The determination of the maximum and minimum moisture contents which may be controlled in the subgrade soil by proper subgrade design in the various climatic regions of the country.

The determination of the test limits for good and bad subgrade soils for the various regions of the country, where climatic, soil, and traffic conditions are similar.

The determination of the test limits of soils, with varying degrees of permeability, which establish whether tile drains are unnecessary, effectual, or worthless.

The discovery of the stages which accompany the phenomena of freezing of soils under existing pavements.

LABORATORY AND FIELD TESTS NOW IN USE

A number of laboratory tests have been devised by the Bureau of Public Roads.[6] These include the method of preparation of the soil samples; the determination by a mechanical analysis of the various percentages of sand, silt, and clay in the soil; the moisture equivalent test for the amount of water the sample will hold against 1,000 times the force of gravity; the capillary moisture test for the amount of capillary water the sample will take up under prescribed conditions; the volumetric shrinkage test for the amount the soil will shrink when oven-dried after being wetted to capillary moisture saturation; the comparative bearing value test to determine the relative bearing power of different soils with desired amounts of water; the slaking value of the soil or the quality of deliquescing in the presence of water; the dye adsorption test which is a measure of those properties of the soil that make for lack of stability. The interpretation of the tests for any given sample must be very general unless all the conditions existing in the field are known. In general, however, it may be said that soils with a high percentage of clay show a high moisture equivalent percentage, a high capillary moisture capacity, a high volumetric shrinkage percentage, a low comparative bearing value, a long time for the slaking value, and a high dye adsorption percentage.

Field methods[7] for identifying good and bad subgrade soils consist in the use of United States Bureau of Soils survey bulletins to determine the general characteristics of the soil, checked by establishing the lineal shrinkage percentage and moisture equivalent by methods outlined by the Bureau of Public Roads.

THE OUTSTANDING FINDINGS OF PAST AND PRESENT SUBGRADE STUDIES

Early subgrade investigations.—Intensive studies of the character of the subgrade to determine the relative stability of various soil types for foundations for road surfaces have been initiated only within the last five years. But even at this late date it is still a common occurrence to see the safe load to which foundation material may be subjected expressed somewhat as follows:

Safe load per square foot

	Tons
Ledge rock	36
Hardpan	8
Gravel	5
Clean sand	4
Dry clay	3
Wet clay	2
Loam	1

The limitations of these figures are usually pointed out by calling attention to the many kinds and mixtures of the materials and the impossibility of applying specific rules to all cases. It is usually further emphasized that an ample and varied experience in foundation work is necessary before any definite allowable pressures for the foundation can be selected. These qualifications make it plain that at best only an approximate estimate of the bearing power of the soils can be made because of the lack of test data upon which more precise estimates may be formulated.

In the early years of highway construction in this country these figures for the bearing power of foundations for bridges were assumed to be applicable to highway subgrades in lieu of more pertinent information. In a vague way this grading did represent the relative bearing power of the subgrades but a more critical analysis in the hope of a more detailed differentiation led highway engineers to the conclusion that below the sands and through the clay there is a wide difference in soils caused by their character as well as the size of the soil particles. This difference affects the amount of swelling or shrinking of the soil with variations in moisture content and apparently also the amount of heaving when subjected to freezing.

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[5] Many soils make good subgrades which might not meet the requirements of a sand-clay or topsoil road.

[6] Procedure for testing subgrade soils, by J. R. Boyd, PUBLIC ROADS, vol. 6, No. 2, April, 1925.

[7] Practical field tests for subgrade soils, by A. C. Rose, PUBLIC ROADS, vol. 5, No. 6, Aug. 1924.

Field methods used in subgrade surveys, by A. C. Rose, PUBLIC ROADS, vol. 6, No. 5, July, 1925.

The California study of the Bureau of Public Roads.—The question immediately arose as to what characteristic of a soil determined whether it would make a good or bad subgrade. One of the first attempts to solve the problem in a scientific manner was made in connection with the study of the California highway system by the United States Bureau of Public Roads in 1920.[8] The soils in this report were classified as follows:

(1) Clay and adobe soils (includes clay, silty clay, clay-loam, and clay).

(2) Marsh lands (includes salt, marsh, and peat lands).

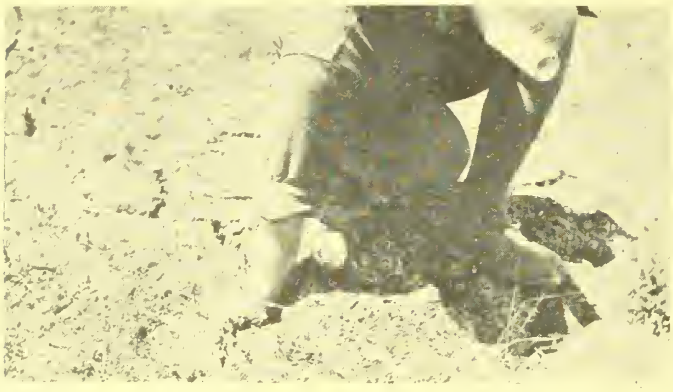

FIG. 1.—Shrinkage cracks in the adobe soil in the open field adjacent to the Pittsburg test road were large enough to admit the arm halfway to the elbow

(3) Loams (includes loam, clay-loam, silt-loam, and silty clay-loam).

(4) Sandy loam (includes coarse sandy loam and fine sandy loam).

(5) Sand, and sand and gravel.

Crack surveys were made of the pavements and on this basis they were classified as to their relative condition. The report concluded that 70 per cent of the defective pavements occurred on adobe soils. A typical failure on the adobe soils consisted of the drying out and shrinking of the subgrade under the edge of the slab because of the evaporation due to the hot, dessicating summer winds. Traffic then broke down the unsupported cantilever of the pavement with the result that longitudinal cracks at the third point of the pavements were characteristic for long distances. As stated by the report, "Typical longitudinal (and other) cracking found on adverse subgrade soils, and shown by many of the 7,500 photographs now on file in the Bureau of Public Roads, indicates a distortion of the subgrade due to varying moisture content and shrinkage. The diagrams showing lines of equal moisture content clearly indicate the influence of the concrete pavement in preventing evaporation. The high capillarity of adobe soils and the great shrinkage in the long, hot summers thus produce very unfavorable conditions for a thin pavement under increasing traffic."

While this report recorded a decided forward step in the development of a classification of subgrade soils, it did not point out the fundamental characteristics of the soil which determined whether it would make a good or bad subgrade. The tests made on the soil samples included the standard moisture equivalent test used by Briggs and Shantz and volumetric shrinkage tests with the samples wetted to about capillary saturation. An analysis of these shrinkage tests shows that in general the values increase with the moisture equivalent percentage but there is a wide range in the values for the same percentages of moisture.

The progress in subgrade investigation which marked the California study may be summarized as follows:

(1) A definite classification of soils with respect to their suitability for highway subgrades was made upon the basis of soil types.

(2) Tests were made to determine the moisture content, moisture equivalent and volumetric shrinkage percentages of subgrade soils.

(3) The moisture equivalent percentage was indicated to be a critical value in respect to the bearing power of the soil.

(4) Seventy per cent of the defective pavements were shown to have occurred on the adobe soils. (fig. 1.)

(5) Typical longitudinal (and other) cracking found on adverse subgrade soils indicated a distortion of the subgrade due to varying moisture content and shrinkage.

The California study did not suggest any method for identifying poor subgrade soils nor did it develop any means of identifying the fundamental characteristics of a soil which determine its value for a highway subgrade. It did give, however, observed test values for subgrades which had been previously expressed only as generalities or at best in a much less specific manner. Of the earlier contributions to the subject, one of the most valuable was the excellent paper entitled "Water and the Subgrade."[9] The writer outlines in detail the necessity and reasons for adequate subgrade drainage, shows that capillary water, because it is unaffected by gravity can never be removed by tile drains, describes what may possibly be the natural processes which take place when soils freeze, and among other things urges the necessity of testing subgrade soils and using in the subgrade only those soils which meet certain recognized requirements. One of the most important statements in the article is that: "The obvious remedy for this condition of affairs (the plasticity or inadequate bearing power of clay soils with capillary moisture present) is to use less clay in highway subgrades."

THE MOISTURE EQUIVALENT PERCENTAGE CRITICAL IN RESPECT TO BEARING POWER

In the California study samples of the soil containing varying percentages of moisture were tested in the laboratory of the Bureau of Public Roads with uniformly increasing loads applied through a bearing block 10 square inches in area and the corresponding penetrations were measured. The statement was made that "* * * the moisture equivalent percentage is a critical percentage in respect to bearing power." While this is in accord with the other findings of the Bureau of Public Roads at the Arlington Experimental Farm,[10] it does not mean that soils with a moisture content in excess of the moisture equivalent percentage do not have sufficient bearing power to support reasonable loads. It does mean, however, that the bearing power of the soil falls off rapidly when the moisture equivalent percentage is exceeded. A more recent determination of this characteristic by the Bureau of Public Roads may be seen in Figure 2. The soil sample has a moisture equivalent percentage of 25.4. With increments of moisture up to 25.5 per cent there is very little increase in the penetration. An additional 4.3 per cent, however, which makes a total of 30.8 per cent of moisture, makes for a decided increase in the penetration, and further increments of water cause the penetration to increase rapidly.

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[8] Report of a study of the California highway system by the United States Bureau of Public Roads. Issued 1920. Revised 1921. Government Printing Office.

[9] Water and the subgrade, by J. L. Harrison, PUBLIC ROADS, vol. 1, No. 12, April, 1919.

POSSIBILITY OF CONTROLLING MAXIMUM MOISTURE CONTENT IN A SUBGRADE SOIL

The field investigations of the Bureau of Public Roads indicate that there is a possibility of controlling the moisture content of a subgrade soil to the moisture equivalent percentage as an approximate maximum value. Observations in the Pacific Northwest [11] generally indicated that the moisture content percentage was greater than the moisture equivalent percentage in the subgrade soils under recent failures in pavements but that the reverse was true when the pavements were in good condition. It was also found that a Cove clay soil, in the open field before any pavement had been laid and where the surface drainage was good, showed a moisture content less than the moisture equivalent percentage at a point 6 inches below the surface (to offset the effect of surface water) and immediately after heavy winter rains. It was considered possible, therefore, that a condition which existed naturally might be reproduced in a subgrade soil by a proper pavement and subgrade design. The natural soil seems to absorb water up to the moisture equivalent percentage and the balance runs off if the surface has sufficient slope for drainage. On the contrary, the pavement when placed on a soil acts as a roof which sheds rain water that does not leak through the cracks which develop in the pavement. The pavement, however, retains two kinds of water which wet the subgrade from below: (*a*) Water of condensation which is precipitated upon the lower side of the slab and falls back upon the subgrade; (*b*) water derived from "needle ice" which melts and runs over the subgrade. It seems hopeful that a pavement design may be adopted which will prevent an accumulation of water in the upper layer of the subgrade soil which is in excess of the moisture equivalent percentage.

The investigations of the bureau indicate that the subgrade soil at depths below the surface sufficient to eliminate the effect of surface water, generally has a maximum moisture content throughout the year which does not exceed the moisture equivalent percentage. The importance of this possibility of water control in the subgrade is apparent because the moisture equivalent percentage also seems to be the critical percentage in regard to the bearing power of a soil. Sufficient data have not been collected to formulate this as a general observation for the country at large but it has been found to be generally true in the States of Oregon and Washington where the freezing temperatures are infrequent, in Minnesota during freezing conditions, and at the Arlington Experimental Farm in Virginia where alternate freezing and thawing occurs. In many of these cases, however, the surface of the poorly drained subgrade soil was found to contain moisture in excess of the moisture equivalent percentage and even to complete saturation. The California study also was significant in showing the moisture equivalent percentage as the maximum for the subgrade soil in the lower depths. The moisture contents seemed to increase with the moisture equivalent percentages of the soil. Of 103 soil samples which were tested, only seven samples showed a field moisture content in excess of the corresponding moisture equivalent percentage and in only one of these cases was this difference greater than 2½ per cent. This applied to samples taken near the surface as well as at considerable depths. These investigations were made during the summer months and there is no direct evidence to show that the maximum moisture content during the winter did not exceed the moisture equivalent percentage in a number of cases. In the California study, however, 49 of the 103 soils were sampled at a depth of more than 3 feet. Although the excessive evaporation in California would tend to increase the depth beyond which the moisture content remains uniform, the comparison of these figures should be interesting. Of these 49 samples, in only 3 cases did the moisture content exceed the moisture equivalent percentage. In one case the excess was 2.3 per cent, in another 2.2 per cent, and in the last 1.4 per cent. The California study lends strength to the observation that the moisture content of a well-drained subgrade may possibly be controlled below the moisture equivalent as a maximum.

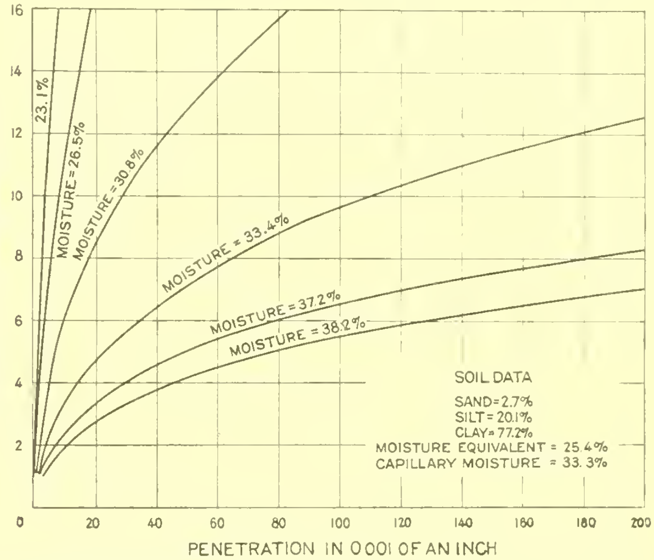

FIG. 2.—The moisture equivalent percentage seems to be a critical value in respect to bearing power. The bearing power falls off rapidly when the moisture content is made to exceed the moisture equivalent percentage

Experiments were made by the Bureau of Public Roads [12] to determine whether it was possible to control the moisture content of a subgrade by various means, such as tile drains and cut-off walls and various subgrade treatments such as gravel subbases and water-gas tar. These experiments apparently yielded negative results although they confirmed the findings of other experimenters that tile drains remove free water from the subgrade but not capillary water. The observed moisture contents of the first four of the 10 treated subgrades throw light upon the possible maximum seasonal limit of the moisture content of a subgrade, although moisture equivalent values were made only upon similar samples taken at the sides of the slabs and not upon the same samples for which moisture contents are shown.

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[10] Physical properties of subgrade materials, by J. R. Boyd. Proceedings of the American Society for Testing Materials, vol. 22, 1922, Pt. II, pp. 352–353.

[11] Practical field tests for subgrade soils, by A. C. Rose, PUBLIC ROADS, vol. 5, No. 6, August, 1924.

[12] Subgrade drainage tests yield interesting preliminary data, by Ira B. Mullis, PUBLIC ROADS, vol. 4, No. 6, October, 1921.

In the first four cases there was used either no treatment as in the first, a subbase combined with tile drains as in the second case, or gravel subbase alone as in the third and fourth sections. In the first case without tile drains or subbase the maximum moisture content 6 inches down from the base of the pavement was 23.4 per cent. Moisture equivalent percentages adjacent to the same slab varied from 16.2 to 17.4 per cent and capillary moisture percentages from 26.6 to 35.7 per cent. In the second case the maximum moisture content 6 inches down from the base of the subbase was 22.6 per cent and moisture equivalent values in the nearby soil varied from 17.0 to 20.9 per cent and the capillary percentage from 26.2 to 35.4 per cent. In the third case the maximum moisture content 6 inches down from the base of the subbase was 21.8 per cent and moisture equivalent values in the soil adjacent to the test slab were 13.5 to 21.5 per cent and capillary percentages from 33.3 to 41.9 per cent. In the fourth case the maximum moisture content 6 inches down from the base of the subbase was 23.7 per cent and moisture equivalent values made, as in the first three tests, varied from 13.2 to 18.4 per cent and capillary percentages from 31.2 to 32.4 per cent.

Only in the case of the untreated subgrade did the maximum moisture content at the 6-inch depth exceed greatly the moisture equivalent percentage and even in this case, as in all the others the maximum moisture content of the subgrade during the entire season was closer to the moisture equivalent percentage than to the capillary percentage. Six inches below the subbase was considered representative in order to avoid the effect of surface water seeping under the pavement and excessively wetting the upper layers of the soil. The effect of frost in causing an increase in the moisture content of the soil near the under side of the pavement is evidenced by the tests only in the first case. The results strongly indicate that it should be possible to control the maximum moisture content of the subgrade very closely to the moisture equivalent percentage by subbases or tile drains.

STABILITY RATIO AS AN INDEX OF BEARING POWER

Defining the term *stability ratio* as the actual moisture content percentage of the soil divided by its moisture equivalent value, the field investigations of the Bureau of Public Roads in Oregon and Washington seem to indicate that the bearing power of a soil is relatively low when the stability ratio is greater than unity and, conversely, that when the stability ratio is less than unity, subgrades are generally well compacted, firm, and hard. The application of this criterion is limited to soils with moisture equivalent percentages greater than 20, but such soils have been shown by investigation to be, in general, the only ones of the suitability of which there is any doubt.[13] Whether the stability ratio is more or less than one can be easily determined by the ease or difficulty with which a drill penetrates the soil. When less than unity, the drillings come from the ground in hard, shiny spirals; when more than unity, the soil is generally soft and sloppy or puttylike, and the drill may be pushed down without rotation. Under new pavement failures in the Pacific Northwest the stability ratio was found to be almost invariably more than unity. This observation is full of potential possibilities. Certainly a marked advance in the progress of highway subgrade evaluation would be made if it were definitely shown that the maximum moisture content of a well-drained soil could be controlled and maintained below the moisture equivalent percentage at all seasons of the year.

It is interesting to note that a relation quite similar to the stability ratio was established in the subgrade laboratory of the Bureau of Public Roads and called the *moisture index*.[14] The relation was expressed as follows:

$$(1)\ \text{Moisture index} = \frac{\text{volume of moisture}}{\text{volume of voids in soil}}$$

Based upon data determined in the laboratory, it was found that when the moisture index of the soil is less than unity, the bearing value was high, but low when the relation was more than unity except in instances where the dye absorption number of the soil was high.

It should be interesting to compare the equation (1) with the stability ratio, which is expressed as follows:

$$(2)\ \text{Stability ratio} = \frac{\text{moisture content percentage}}{\text{moisture equivalent percentage}}$$

THE RECENT TREND OF ENGINEERING OPINION

It might be well to add here that for many years engineers have recognized the adverse effects of clay in subgrades without evolving any adequate method to evaluate the character and quantity of the clay present, or to determine the degree to which it would affect the stability of the road surfacing, or without attempting to arrive at an adequate surfacing or pavement, subbase or drainage design to overcome the trouble. Citations to the mass of data which have been piling up throughout the years as evidence of the effect of the character of the subgrade are given here briefly to show that these observations are not original with any one engineer or any group of engineers nor are they confined to any single period. On the contrary, they have been made by representative members of the engineering profession throughout its entire history and, therefore, the composite conclusion which may be drawn from the testimony is based upon comprehensive as well as authentic information. For brevity and completeness only recent data will be referred to.

Life of railway trackage affected by character of subsoil.—An abstract of a report to the joint committee on life of railway physical property of the American Electric Railway Engineering and Accountants' Associations[15] presents some instructive data concerning soil as well as climate and their effect upon the foundation and drainage of railway property. "On no two separate properties are climatic and soil conditions so similar that the physical elements of such properties would not be affected differently. * * * Property in a mild climate would not be affected by the conditions met with in a cold climate. Wet and dry climates affect physical properties differently, as also do the different soils of different localities. Construction on a rock foundation has a physical life materially different from the construction on any less firm foundation. Two localities may have the same soil, but one is situated topographically so that it is easily subjected to adequate drainage, while the other may be almost impossible of drainage owing to its topography. This would also affect differently the physical life of each. Any attempt to establish the life of physical railway property based on climatic and soil conditions would result in as many different periods of life as there are properties under consideration."

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[13] Practical field tests for subgrade soils, PUBLIC ROADS, vol. 5, No. 6, August, 1924.

[14] Physical properties of subgrade materials, by J. R. Boyd. Proceedings of the American Society of Testing Materials, vol. 22, 1922, pt. 2.

[15] R. N. Wallis, Fitchburg & Leominster Ry., Fitchburg, Mass., and Martin Schreiber, Public Service Ry., Newark, N. J., chairmen. Presented at the annual convention in Atlantic City, N. J., Oct. 16, 1913. Engineering News, vol. 70, No 20, Nov. 13, 1913, p. 972.

Sand subbase prevents soil from working up into voids of macadam.—Another adverse effect of clay was brought out in a discussion before a meeting of the Boston Society of Civil Engineers, by John R. Rablin, chief engineer, park division, Metropolitan District Commission of Massachusetts.[16] "We have found, where we have a clay subgrade, that if we lay crushed stone or coarse gravel over it before laying our macadam, in a few years, if we have occasion to excavate, most of the stone seems to have disappeared and the clay to have come to the surface or mixed with the stone. This action is probably due to frost and moisture. We have also found that by covering the clay with a layer of fine sand before putting on the coarse gravel or stone, it will prevent that to a large extent." Here are data which not only describe a condition but also advocate a sand subbase as a method of correction. Experiments of the Bureau of Public Roads at the Arlington Farm confirm this as a practical subgrade treatment.

Cracking of pavements due to shrinkage of adobe soil.—The experience of the city of San Antonio, Tex., in laying pavements upon what appears to be adobe soil is of interest.[17] "The subsoil is a sticky, black material when wet and it cracks in drying, with the result that the concrete usually cracks too. When the dried soil again takes up water it swells an appreciable amount. Successive wetting and drying enlarge even a hair crack in the pavement surface into wide breaks * * *." This exemplifies the shrinking and swelling of heavy clay soils with variations in moisture content. A similar condition was noted by the writer in the city of Rio Vista, Calif.

Excessive heaving by frost in clay soils.—The excessive heaving in clay soils due to freezing is outlined in an article by Stephen Taber[18] in which it is stated that "in a middle west city 16-ton concrete piers resting on 'solid blue clay' were elevated 3 inches during the unusually severe winter of 1916–17, and that subsequently they settled back to their original position." The theory had been advanced that this lifting was due to the hydraulic pressure resulting from the progressive freezing of the ground from the surface downward. Since the frost could not have penetrated more than 6 inches to 1 foot below the bottom of the piers and the expansion of this depth of water would not be sufficient to account for the 3-inch heave, Mr. Taber points out that this theory is inadequate because the specific gravity of the concrete is greater than that of the earth and, therefore, if there were any hydraulic pressure it would be expected that the frozen crust of the earth would be forced up rather than the concrete.

Mr. Taber states that the hypothesis of the soil being acted upon by a force similar to hydraulic pressure does not offer an adequate explanation of the phenomena. On the contrary he explains the case on the basis of his experiments which showed the pressure[19] exerted by growing crystals of ice. The following excerpts from his discussion are of singular interest to highway engineers:

"In the course of some of the writer's experiments, carried out on cold nights, weights placed on wet clay were lifted through the gradual formation of virtually pure ice between the weights and the clay. Weights that were similarly placed on wet sand were not lifted, or at least their elevation was imperceptible, and no pure ice was formed under the weights, although the water occupying the interstices between the sand grains was frozen.

"Water in extremely small capillary spaces remains in a liquid state at temperatures much below freezing. Most of the interstices in clay are in the form of very small capillary spaces, while in sand they are much larger. Therefore, the interstitial water freezes much more readily in sand than in clay.

"In the experiments referred to above the weights cooled off rapidly, thus chilling the film of water in contact with their lower surfaces until ice began to form. Where the weights rested on wet sand freezing gradually continued downward, thus firmly cementing the sand grains together. The grains of sand were slightly separated in places as a result of the freezing, but the total elevation of the mass was so slight as to be imperceptible. Where the weights rested on wet clay, however, the ice forming at the surface gradually increased in thickness because of the freezing of additional water that slowly reached the surface of the clay through the small capillary openings. Because of its manner of growth the ice had a fibrous texture, and as additions were made to its lower surface the ice was slowly elevated, together with any weights resting upon it.

"The ice columns, or 'needle ice,' formed on bare clay soils are familiar to most people living in regions where the nights are cold enough for heavy frosts. These ice columns, which commonly support sand grains, pebbles, and sometimes larger objects, are formed in the manner explained above; and all of the facts cited * * * indicate that the elevation of the concrete piers was probably due to this same process. If the piers had rested on sand instead of clay it is improbable that any elevation would have been noticed, no matter how thoroughly saturated with water the ground may have been.

"The elevation of the weights in the writer's experiments is not to be explained by attributing it to the expansion in volume, which occurs when water freezes to form ice, for the distance through which the weights were lifted was approximately equal to the total thickness of the ice. The lifting of the weights was due to the growth of the ice crystals, and this would continue in spite of the pressure as long as water was available for growth and the temperature was low enough to cause freezing. With any increase in pressure, however, a lower temperature would be required in order that freezing should continue, for the melting point of ice is raised by increasing the pressure or in any way placing the ice under additional strain." These very interesting observations of Mr. Taber's indicate strongly that the greatest heaving due to freezing occurs on clay soils.

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[16] A detail of road foundations in clay, Engineering News-Record, vol. 88, No. 11, Mar. 16, 1922.

[17] Texas soil requires paving base to be reinforced, Engineering News, vol. 75, No. 14, Apr. 6, 1916.

[18] State geologist and professor of geology, University of South Carolina. "Ice forming in clay soils will lift surface weights," Engineering News-Record, vol. 80, No. 6, Feb. 7, 1918.

[19] Pressure phenomena accompanying the growth of crystals. Proceedings of the National Academy of Sciences, vol. 3, pp. 297-302, April, 1917.

Frost-heaving phenomena of two classes.—These experiments, however, do not offer an explanation of the phenomena of heaving under pavements. There are two general types of these phenomena. The first is a progressive lifting of the pavements as the winter advances and the penetration of the frost increases in depth. This happens principally in sections of the country where extremely cold winters are the rule, such as Minnesota, and where the thermometer only occasionally rises above the freezing point after the cold weather has set in until the spring thaw begins. In this case the pavement gradually subsides into its original position when the frost gradually leaves the ground, and if the upward movement has been uniform and there has been no warping of the surface there seems to be little cracking due to the freezing. Unfortunately this uniformity of subgrade is rarely found and warping often occurs. For this reason cracking is generally the rule under this condition of heaving. The second condition is found in the Eastern States where the temperature throughout the winter rises and falls periodically above and below the freezing point. After a long thaw and usually in the spring there may come a severe drop in temperature. The result is that freezing occurs first in the earth shoulders between the ditches and the pavement and seems to progress inward toward the center of the road. It seems possible that the free water and moist subgrade under the center of the pavement is the last portion to freeze and in expanding rapidly exerts pressure upon the surrounding subgrade and the pavement above. Since the subgrade is more stable than the pavement the latter is heaved and broken badly over night. Observers who have seen this condition testify to mounds of ice which have been seen under the pavement at the heaved sections. These mounds are believed to have been caused by the outrushing water from under the pavement which froze immediately after the pavement was heaved and the pressure released.

Examples of the first or gradual type of heaving seem to have been furnished by the observations on existing pavements in Minnesota made by the Bureau of Public Roads which showed the progressive lift of the pavement at the end of the season to have reached a maximum height in some cases of 7 inches.

The condition of rapid heaving is ably discussed in a paper published in the Engineering News-Record.[20] "If the pressure is relieved by an outpouring of water only," according to this writer, "the pavement will settle back in place when the ground thaws out in the spring. Unfortunately, however, the water often carries with it enough solid matter so that the bunch formed at the point of rupture will not completely disappear."

Experiments by the Bureau of Public Roads show that there is a greater percentage of water freezable in sands than in clay.[21] This does not mean that the amount of heave in sands due to freezing will be greater than in clays. On the contrary, it would seem that the greater heaving would occur in clay with capillary water present because of the greater total amount of water frozen. Further experiments are needed to determine the relative heaving of coarse and fine-grained soils with varying moisture content.

Vertical displacement of concrete pavements by frost heaving of subgrade.—In a paper advocating the use of cut-off walls under the edge of a pavement to control the variation in moisture content of the subgrade, given before a convention of the American Concrete Institute, John W. Lowell, division engineer of the Universal Portland Cement Co.,[22] makes some interesting observations with regard to the effect of heaving on the pavement. Investigations of the Bureau of Public Roads throw considerable doubt upon the efficacy of cut-off walls in controlling the subgrade moisture content; but the data presented with respect to heaving have considerable value, especially the following quotations:

"In climates where freezing occurs, it has been found from general observation of pavement slabs less than 60 feet in length, that the following is true:

"(1) Transverse cracks do not occur as frequently as longitudinal cracks.

"(2) For width over 18 feet, frequency of longitudinal cracks is apparently unaffected by width of pavement.

"(3) Cracks seldom occur in pavement lying on sand or well-drained porous subgrades.

"(4) On clay or heavy loam soils, longitudinal cracks are of frequent occurrence.

"(5) The more compact the subgrade soil, the more frequently cracks occur.

"(6) Cracking of pavements on clay or heavy loam soils can not be entirely prevented by artificial drainage."

A section of concrete pavement 32 feet wide, 10 inches thick in the center, and 7 inches thick at the sides was selected to make the tests. The base was 1:2½:4 pebble concrete and the wearing surface 1:1:1½ concrete. The reinforcement consisted of No. 27 triangular wire mesh fabric weighing 41 pounds per 100 square feet placed between the base and the wearing coarse. Precise levels and data on temperature and rainfall were taken. "From slab 1 to 38 the subgrade may be described as spongy, tough, compact clay; from 39 to 45, sand-loam with spongy understratum and compact crust; from 56 to 70, clay and loam with some fine sand, the ground being spongy with sand below; and from 79 to 86, the south end of the pavement, sand of more than 3 feet depth. * * * Out of 86 slabs, 56 cracked. Thirty-eight which cracked deflected at one or both points more than seven-sixteenths inch in the 32-foot width between outside points, 18 deflected less than seven-sixteenths inch and of these deflections eight were between three-eighths inch and one-sixteenth inch, while four slabs having as great deflection failed to crack. Six of the

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[20] First frost is never responsible for cracked concrete roadways, by J. L. Harrison, Engineering News-Record, vol. 80, No. 9, Feb. 28, 1918.

[21] Percentage of water freezable in soils, by A. M. Wintermyer, PUBLIC ROADS, vol. 5, No. 12, February, 1925, p. 5.

[22] Impervious Bituminous Wall Suggested to Prevent Seepage Under Paving, Engineering News-Record, vol. 81, No. 5, Aug. 1, 1918.

10 remaining slabs which cracked, having maximum deflection of from three-eighths to three-sixteenths inch, evidently cracked from abnormal settlement of a portion of the subgrade below the original level. The cracking of four slabs, with maximum deflections between three-sixteenths and five-sixteenths inch, is hard to explain unless the deflection had been greater at some period between readings. Every slab reaching a deflection of more than seven-sixteenths inch cracked. Therefore, it is safe to assume that such deflection in 32 feet of width is sufficient to crack any pavement built in accordance with present specifications. It is also evident that even greater deflection than seven-sixteenths inch can be expected when the subgrade is clay or loam with no other protection than underdraining. From study of the data collected it was shown that practically no longitudinal cracks occurred where the deflection was less than eleven thirty-seconds inch, and that where the subgrade has a uniform moisture content the deflection would probably never reach this figure, which conclusion is substantiated by those slabs which were over the sand subgrade."

"Thirteen consecutive slabs at the extreme south end of the pavement and on a sand subgrade acted quite differently and more uniformly than the others. The average upward movement was seven-sixteenths inch on the west side and $1\frac{1}{16}$ inches on the east side, while the center rose thirty-one thirty-seconds inch. However, the movements were so uniform that the greatest deflection was one-eighth inch and no cracking occurred. Unlike the other slabs, there was practically no settlement below the initial reading, the greatest settlement recorded being three thirty-seconds inch.

"The movements of two typical slabs, together with the rainfall and temperature, are shown in the charts. (fig. 3.) Upon freezing, the sides and centers of these slabs at first began moving up uniformly. During cold weather considerable snow fell, and then in the first week of January a thaw occurred, followed by extremely low temperature for the next eight weeks. In this period the sides of slab 18 on clay subgrade rose more rapidly than the center, until a maximum deflection of five-eighths inch occurred, while slab 86 on sand subgrade rose only six-tenths as high, there being practically no deflection. These concrete slabs were exactly alike, and precipitation and temperature were the same, the only difference being in the character of the ground on which they lay."

Mr. Lowell's explanation of the phenomena of heaving follows: "Upon freezing, the soil expands in proportion to the moisture contained, and the greater the difference in moisture between the sides and the center the greater the distortion of the pavement slabs. When a thaw occurs, if the soil is not fully saturated, the water sinks and spreads out under the pavement as far as it can go by virtue of head and capillary power. This moisture settles in until it freezes in the voids at some depth where the temperature is still below freezing. In this way an impervious stratum is formed through which the water will not pass. The later seepage is retained above this stratum, and the soil rapidly becomes more water soaked. With the return of colder weather and subsequent freezing of all moisture this subsoil expands, and, being restrained laterally, the pavement is heaved. Naturally, the vertical movement or heaving is greatest at the sides where moisture is at a maximum."

Mr. Lowell brings out clearly the amount of heaving due to freezing in a soil of high clay content, describes accurately the increase in vertical movement with a progressive drop in temperature, and shows that a center deflection of seven-sixteenths inch in the concrete slab 32 feet wide is sufficient to cause a longitudinal crack to form. The explanation of the phenomena of freezing, while probably sufficient for this particular problem, does not explain the matter satisfactorily where physical conditions are different and where the heaving is by a rapid rather than by a slow progressive lift.

It is unfortunate that most of the articles bearing upon the subject of heaving of pavements are discussions of the possible process of the heaving based upon experiments made in connection with other structures than pavements or are discourses premised upon heaving data rather vaguely expressed by a second party. At this time there seem to be no authentic data available as to the progressive stages in the freezing of the subgrade and the heaving of the pavement. The problem will never be satisfactorily solved until some research organization carries on a series of intensive studies on existing pavements. Such studies should include the raising of the pavement slabs and the making of photographs of conditions underneath throughout the winter when the frost lift is gradual, and up to, through, and after the period when the thaw causes the break-up of the pavement. Precise level readings at all stages would be invaluable. Until

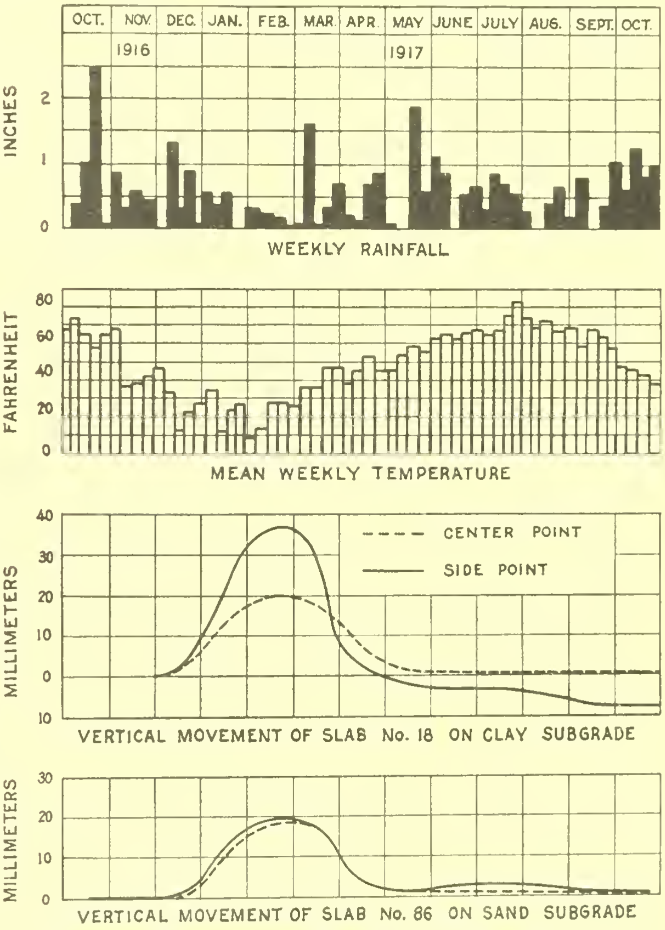

FIG. 3.—Climatic statistics and comparison of the vertical movement of a concrete pavement on clay and sand subgrades due to freezing

some work of this character is carried on there seems little hope of arriving at an intelligent or satisfactory solution of this problem.

Operation of tile drains affected by permeability of soils.—The permeability of soils is discussed in an interesting article[23] in which it is shown that the porosity of the soil increases with the size of the grains. *M* is defined as "the quantity of water that would be transmitted in a unit of time through a cylinder of the porous medium, one unit in length, one unit cross-sectional area, and under one unit head or difference of pressure at the ends of and just inside the cylinder. *M* may be called the 'modulus' of the medium. * * *." The following figures are due to Professor Nazzani, of Rome:

Kind and size of material—Values of M in 24 hours

	Cubic feet
Small gravel (size about 2 mm.)	2, 075
Coarse sand (size about 2 mm.)	224
Fine sand (size about 0.2 mm.)	40
Sandy soil	19
Sandy clay	10
Clay	2. 8

The adequacy of tile draining for subgrades has been discussed pro and con by highway engineers for some time. The opinion now seems to prevail that there are certain soils such as sands in which tile drains are unnecessary; there are others in which the porosity of the soil is sufficient to permit effectual operation of the tile drain, and others, principally the clays, in which the porosity seems to be so small as not to permit sufficient water to enter the tile drains to make them effective. There seem to be special cases in clay soils, however, where tile drains are valuable. Examples are drains installed on the uphill side of long slopes to prevent the softening of the subgrade by standing surface water, water in soil pores or seams, seepage, porous strata, and hydrostatic pressure. It is evident, however, from Professor Nazzani's table that it should be possible to determine the soil types in which tile drains would be effective by carrying on experiments on soils with different sizes of grains, the character of the grains remaining constant.

Bearing power of soils.—The relative bearing power of soils has been demonstrated by a number of tests made by the Bureau of Public Roads.[24] Curves are shown with the load in pounds plotted as ordinates and the penetration in inches as abscissae for six soil types with moisture equivalent values of 2, 10, 19, 23, 34, and 41 and respective percentages of clay by mechanical analysis of 2, 19, 26, 26, 38, and 57. In order to compare the relative bearing power of these soils with the sizes of the grains increasing in fineness from one sample to another, it would be necessary to select arbitrary loads in pounds with the soils wetted to the moisture equivalent percentage. Upon such a basis these test results show no increase in bearing power with decrease in the clay content. This should not be accepted as a definite conclusion, because the tests were limited in their scope and intended to explain the method of soil testing rather than to present experimental data as to the relative bearing power. A part of the article, however, is considered of sufficient significance to warrant quotation here: "Results of the tests would indicate also that the bearing power of most soils is not appreciably reduced by the addition of moisture up to the moisture equivalent. * * * When the soils become saturated beyond the moisture equivalent, however, a rapid reduction apparently takes place in bearing power, which usually disappears completely at or before the water-holding capacity of the soil has been reached. These general comments are given here simply to indicate possible existing relations and are in no sense definite conclusions, which manifestly can not be drawn without much additional study."

Depths of macadam varied with subgrade and height of fill. An interesting contribution relative to the bearing of the character of subgrade soil, the depth of fills, and the density of traffic on the required depth of macadam surfacing was published by the Superintendent of Roads of Monroe County, N. Y., in 1920.[25] The data are not sufficiently detailed to be of any great value at the present time for design, but the method of attacking the problem shows the trend of thought toward a selection of surfacing based upon traffic, soil, and other subgrade conditions. The third variable, climate, which is essential for evaluating designs over the country at large, was omitted since the area involved was only New York State, where the climatic variations were relatively small. Table 1 and its accompanying remarks are quoted from the article.

TABLE 1.—*Recommended depths in inches of macadam for different conditions*

Class of traffic	Subgrade soil	Depth of macadam in cuts	Depth of macadam in fills		
			Depth of fills		
			Less than 1 foot	1 to 3 feet	Over 3 feet
		Inches	*Inches*	*Inches*	*Inches*
Class I	Coarse sand and fine gravel	9	9	9	9
	Clay-loam	12–15	12–15	10–12	10
	Heavy clay and quicksand	20–24	20–24	12–18	10
Class II	Sand and gravel	7–8	7–8	7–8	7–8
	Loam	9–12	9–12	9	8
	Clay and quicksand	15–22	15–22	12–15	9
Class III[1]	Sand and gravel	5–8	5–8	5–8	5–8
	Loam	8–12	8–12	8	7–8
	Clay and quicksand	15–22	15–22	10–15	9

[1] Under Class III, it is better to use the depth recommended for Class II and reduce the width of metalling, if economy is desired, as all improved roads are bound at times to be subjected to the extreme loading produced by heavy trucks.

Class I. Large volume of ordinary vehicles and heavy auto trucks.
Class II. Large volume of ordinary traffic; a few heavy trucks.
Class III. Moderate volume of ordinary farm loads; very few heavy trucks.

The writer recognizes in a general way the necessity for an increase in thickness with increasing clay content of the soil and specifically a decrease in thickness with an increase in the depth of the fill because of the improved drainage conditions.

Hair cracks in concrete pavements on loess soil eliminated by tar paper.—An example of a loess soil causing hair cracks in a cement concrete pavement during the process of curing is reported by R. W. Crum, engineer of materials and tests of the Iowa Highway Commission.[26] The soil runs about 20 per cent clay and 80 per cent silt according to the mechanical analysis of the United States Bureau of Soils. The lineal shrinkage is only 2.3 per cent and the moisture equivalent 21.9 per cent. This is an example of a soil with a small clay content which does not shrink, swell or heave sufficiently to cause extraordinary trouble after a concrete pavement is completed but which causes hair cracks to develop in the concrete pavement during the period of construction. The hair cracking was overcome by interposing a layer of tar paper between the subgrade and the pavement during construction.

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[23] Wells and permeability of soils, from Engineering, London, Dec. 31, 1920. Engineering and Contracting, Mar. 30, 1921, p. 312.

[24] Tests for subgrade soils, by A. T. Goldbeck, PUBLIC ROADS, vol. 4, No. 3, July, 1921.

[25] Variable designs for county highway systems, Engineering News-Record, vol. 84, No. 5, Jan. 29, 1920.

[26] Tar paper on loess subgrade lessens hair cracks in concrete pavement, by R. W. Crum. PUBLIC ROADS, vol. 6, No. 6, August, 1925.

The explanation for the physical phenomena is not given. The angular shape of the grains and the large percentage of voids in a loess soil, may cause rapid capillary action with the result that the water in the fresh concrete is withdrawn before the crystallization of the concrete has reached completion. This would cause a shrinkage of the concrete with the result that hair cracks would develop. Climatic conditions such as wind and a low relative humidity of the atmosphere might contribute to the formation of the hair cracks. The grading and character of the sand and coarse aggregate might also reduce the rententivity of the concrete to water. Whether the cracking in this case is due entirely to soil conditions or to a combination of causes should furnish an interesting subject for further investigation. At least the methods adopted for the Iowa loess soil should not be used elsewhere without thoroughly checking against the local conditions.

It is of prime importance to note at this time, however, that the experiments of Mr. Crum as well as his observations on completed pavements show that the use of tar paper on the subgrade has been effectual in practically eliminating hair cracks.

Selection and redistribution of subgrade soils.—The selection and redistribution of soils to secure a stable subgrade is a possible method of construction. The general practice however is to place a granular subbase upon the undisturbed subgrade soil. This has been accomplished to a certain extent by the method of stage construction practiced in the Southeastern States. There pavements have been constructed on old topsoil roads. The topsoil therefore became a subbase for the pavement or a subgrade with adequate and uniform supporting value which overcame the adverse effects of the bad subgrade soils lying beneath.

FIELD SUBGRADE INVESTIGATIONS OF THE BUREAU OF PUBLIC ROADS

Having presented representative data from various sources to show the trend of engineering opinion on the subject of subgrades, the discussion of the more specific subgrade investigations, begun with the California study by the Bureau of Public Roads, will be continued. The same bureau in April, 1920, began a field inspection of road failures produced by poor subgrade conditions with the idea of determining the proper design of pavement and subgrade. One result of this work has been the development of field tests for subgrade soils and the suggestion of a method for measuring the character of the subgrade.

A field test for the moisture equivalent percentage was developed which gave, for soils with a moisture equivalent percentage greater than 20, identical results with the amount of water retained by a soil sample in opposition to one thousand times the force of gravity by the standard centrifuge method. The test was believed to be valuable for determining the degree of fineness of the soil grains since soils with fine grains would have a larger total surface area than those with coarse grains and therefore the fine-grained soils would generally show a large moisture equivalent percentage. This is not always true because the moisture equivalent percentage varies with the character as well as the size of the grains and, therefore, whether the grains be glazed or rough, solid or porous, will materially affect the result. The character of the surface of the grains may affect the thickness of the water film on each grain; their texture affects their surface area and in that way their capacity to retain water, the more spongy the grain the greater the surface area. The character of the grains affects the moisture equivalent values to such an extent that it has never been found possible to develop an empirical formula for the derivation of the moisture equivalent from the mechanical analysis of the soil separates such as sand, silt, clay, etc. In general, however, the clay content of the soil increases with the moisture equivalent percentage.

The significant test developed in these field investigations was the lineal shrinkage test. It was considered desirable to determine this value because the vertical distortion or displacement of the subgrade resulting from the shrinking or swelling of the soil is a factor which limits the life of a road surface. From an investigation of the condition of a number of pavements coupled with an analysis of their subgrades the conclusion was reached that a lineal shrinkage of 5 per cent, equivalent to a volumetric shrinkage of 14 per cent, would represent the maximum value for a good subgrade soil. Soils with a moisture equivalent of 20 per cent or less were found quite generally to have a lineal shrinkage of less than 5 per cent and were therefore considered as good subgrade soils. Soils with a moisture equivalent between 20 and 30 per cent were classed as doubtful subgrade material because the lineal shrinkage in these cases often exceeded 5 per cent. When the moisture equivalent exceeded 30 per cent the lineal shrinkage generally exceeded 5 per cent, and these were considered bad subgrade soils. These studies were made in the States of Oregon and Washington, and the limits referred to may only be applicable to the soils of that general region. However the significance of the test is not affected by location, and its application and use in other sections would quickly determine the limits which should be set for each region.

From these investigations a method of making subgrade surveys for roads was developed which is varied according to whether the area traversed by the road project is covered by a detailed Bureau of Soils survey map, a reconnaissance map or is in an unmapped area. Where detailed soil maps are available it is only necessary to establish the location of the changes in soil types as shown on the maps with reference to the stationing of the road, and representative samples of the various soil types on the project may then be taken and tested. The results of the lineal shrinkage test are used to classify the soils as good or bad. In the Pacific Northwest, where the method originated a lineal shrinkage of 5 per cent is taken as the limiting value for a good subgrade soil; for other sections it may be necessary to develop other limits. This is the first comprehensive method[27] which has been advanced for the identification of the character of the soil in highway subgrades. It is not original except as applied to highway subgrades, having been used for many years by the United States Bureau of Soils to evaluate the agricultural value of various soil types.

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[27] Field methods used in subgrade surveys, PUBLIC ROADS, vol. 6, No. 5, July, 1925

THE CONTRIBUTION OF THE PITTSBURG TEST

The two great field tests of pavements made at Pittsburg, Calif., and on the Bates Road in Illinois furnished additional but meager information on subgrade soils. The Pittsburg tests were made on a quarter-mile elliptical track containing 13 different types of cement concrete pavement on which traffic began on November 9, 1921; the Bates Road, a 2-mile tangent surfaced with 68 kinds of six major types of pavement, was subjected to traffic initially on March 30, 1922. The first test was carried on by the California Highway Commission and the Pittsburg Steel Co. cooperating with the Bureau of Public Roads. The second test was carried on entirely by the Illinois Division of Highways.

The Pittsburg test road [28] was built on California adobe, a black, sticky gumbo, common to that section of the country. As the construction of the subgrade was considered one of the principal features of the test, the sections of the report pertaining to the subgrade are quoted in part as follows:

"The site of the test highway was a field in which the soil was a very obstinate black adobe. The ground sloped gently from the south to the north and to avoid variations of subgrade condition the entire subgrade was on fill. On the south side, excavation was carried to 3 feet in side borrow pits and the material placed loosely outside and the fill for the entire track was then started approximately 3 feet below finished subgrade elevation. The bottom soil was first plowed about 8 inches to produce a bond, and layers of earth approximately 9 inches in thickness, loose, were successively placed from the borrow across the entire width of roadway, and each layer pulverized by using a disk harrow followed by straight-tooth harrow, and a Johnson scarifier. After pulverization was complete, a light spray of water was applied over each entire layer sufficient to moisten, but not excessively wet the soil, and to aid in compacting it into a dense homogeneous mass. A 12-ton, 3-wheel road roller rolled each layer, and weak spots in the subgrade layers that developed were excavated, and refilled, and rolling continued. The compacted layers were approximately 6 inches thick and each was scarified with a Johnson scarifier for 2 inches to furnish a complete bond with the succeeding layer.

"On this approximately completed subgrade, header boards were set true to line and grade and extending slightly below the surface. Then the material between was scarified to a depth of 6 inches by a Johnson scarifier. The surface was then graded with a Carr subgrade machine to an elevation approximately 1½ inches above subgrade. The surplus earth was temporarily placed on the shoulders. Then the subgrade was scarified to a 4-inch depth and water applied with a very light spray and immediately the subgrade was rolled from the edges inward. While still moist it was again cut with the subgrade machine to the correct elevation. Up to the time of laying concrete the subgrade was given a light sprinkling daily to prevent cracks forming. Where inverted curbs were involved in the design excavations were made with pick and shovel."

Moisture penetration believed combated by subgrade treatment.—The test traffic was begun with the side ditches dry. At the end of the first month (December 21, 1921) the ditches were filled with water to the bottom of the pavements. The report continues:

"This water level was maintained in the ditches until the traffic was discontinued January 30, 1922. However, the water was not drained away until the middle of March, 1922. It was anticipated that with the ditches full of water the grade would soon become saturated and the destruction of the pavement accelerated. The rains were also expected to contribute very materially to this result. This opinion seemed to be shared by visiting engineers who had experience with adobe under such conditions. The result did not meet the expectations. Only a very small amount of additional moisture was taken up by the grade. The moisture determinations of the borings indicated that beyond a lateral distance of 2 feet from the water the absorption was very small. This resistance of the grade to moisture penetration can be attributed to the manner in which it was built."

The foreword of the report points out:

"The satisfactory results obtained by the construction of the adobe subgrade in successive thin, rolled layers, lightly sprinkled, seems to warrant (especially on adobe soil) the additional refinement of this fundamental process, which has been somewhat neglected in recent years. Examination of the final distribution of moisture in the Pittsburg subsoil shows a surprising absence of penetration from the flooded ditches and further studies should determine if such impenetrability is permanent. Therefore this important indicated result of the test should be immediately verified by trials and study in actual construction under equally (or even more) unfavorable conditions."

In describing the moisture contents of the soil the report states:

"Samples of the soil were taken at intervals from the shoulders each side of the pavement and from holes that had been left in the pavement when constructed. Samples were taken of the top layer of soil and also at depths of 1, 2, 3, and 4 feet. These samples were removed from the subgrade by a 2-inch earth auger and immediately were placed in air-tight cans and shipped to the laboratory for moisture content determinations. * * * The variation of the moisture content of the subgrade during the period from December, 1921, to September, 1922, did not exceed 15 per cent."

Discussion of findings.—"The method of constructing the subgrade on adobe soil, as described, so reduced the objectionable features of this material that a reliable foundation for the pavements resulted." This was the first of the seven conclusions of the report. If further investigation proves the adequacy and economy of this method of constructing the subgrade, it should be possible to give it a wide range of usefulness in the adobe areas of the Western States. Its cost and the construction difficulties seem to weigh heavily against it. In California especially, longitudinal cracks at the third point are characteristic of pavements laid on adobe subgrades. These are caused by the adobe drying out and shrinking under the edges of the pavement. The cantilevered slab then breaks off under the traffic. The adobe shoulders on the Pittsburg test pavement were very wide and one foot and more above the top of the pavement in many places. It is possible that this soil layer dried out at the surface but, acting as a blanket, prevented the subgrade under the edge of the pavement from drying out and shrinking. If this observation has any merit it would discount to a large extent the conclusion that the method of preparing the subgrade prevented volume changes in the adobe soil. However, as the report states, the method has potent possibilities which seem to warrant a further and more intensive study of this particular feature.

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[28] Report of highway research at Pittsburg Calif., 1921–22, California State Printing Office, Sacramento, 1923.

BATES ROAD TEST IN ILLINOIS

In discussion of the Bates Road tests it seems best to quote from the report of Clifford Older,[29] under whose direction the test was carried on.

Subgrade believed to be of uniform character.—The uniformity of the soil on the project was described as follows: "Inasmuch as the soil throughout a large part of the State exhibits fairly uniform physical characteristics and considering the fact that the relative behavior of the various sections could not be judged if the nature of the foundation varied materially, a site was selected where subgrade conditions would be as nearly uniform as possible. Practically all unprejudiced observers agree that the Bates Road site fulfilled these conditions as ideally as could be expected on a length of road of two miles or more. No visible variation in the character of the soil could be detected; on the other hand, no positive method of establishing beyond question the lack of variation in subgrade soil could be found."

"Under each edge of a 200-foot section of the Bates Road was laid a tile drain 24 inches below the subgrade, the trench back-filled with cinders, and a free outlet provided for the tile. Moisture samples were taken from the underlying soil at various points throughout this 200-foot section, and likewise from the adjacent undrained slabs. During a period of three years, no measurable difference in the moisture content of the subgrade at these points has been observed. At another place, on the 'Chatham Road,' where clay of a different character is found, tile drains were laid 42 inches under each edge of the pavement for a distance of 1,000 feet, the trenches were back-filled with cinders, and similar extended observations were made, with results as shown on Figure 4. No attempt is made to explain why the soil underneath the section provided with tile drains has throughout the entire period contained more moisture than that under the adjacent pavement. Judging from these two examples in which tile drains were of absolutely no apparent value, it is questionable whether such attempts to control moisture are of any merit whatever in clay soils."

Subsequent moisture content of subgrade affected by amount of wetting during construction.—"It was found that both the brown silt-loam of the Bates Road subgrade and the yellow clay of the 'Chatham Road,' when they have a moisture content which may be considered normal for the summer months, resist further saturation to a marked degree. Attempts to saturate the soil underneath the Bates Road failed. Water standing at subgrade elevation for six weeks during the summer months did not cause a perceptible rise in the moisture content of the subgrade soil at a sampling station 30 inches distant. Tests now under way on 30 samples of clay soils, gathered from different points throughout the State, indicate that this phenomenom in slightly varying degrees probably is common to all.

"Another series of experiments indicate that if the moisture content of any of these clay soils is reduced to a point where the soil is dry enough to crumble readily, absorption takes place rapidly, to the point of saturation.

"These two properties of clay soils may have a marked effect on the supporting power of the subgrade under a freshly laid pavement slab; for example, the sections of the Bates Road built in the fall of 1920 were laid on a sun-dried subgrade, as a rainless period and hot weather had preceded the construction of the pavement. In October, 1920, the first moisture samples were taken from the underlying soil just after a three-day period of rain. At that time the subgrade was found to be as nearly saturated as it ever became. On the other hand, sections 64 to 68, inclusive, were laid in the fall of 1922, when the subgrade soil contained about 25 per cent moisture. The moisture content of the soil under these slabs remained practically constant throughout the winter months of 1922–23 and, although the spring and summer rains were heavy, it continued throughout the summer of 1923 materially below that of the remainder of the road. From this, it may be inferred that for perhaps a period of a year or more after the pavement is laid, the bearing power of a clay subgrade soil may be affected materially by its moisture content at the time of construction."

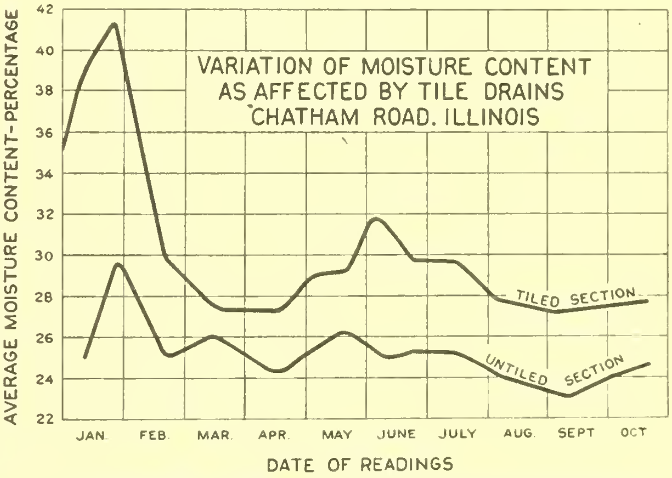

FIG. 4.—No explanation is available to show why the moisture contents of the subgrade on the tiled section were greater than those on the untiled section

In another section of the report the following statement is made:

"Many clay soils when dry, readily take up sufficient moisture to cause a high degree of saturation, but absorb water slowly when moist; hence, subgrades of dry clay should be moistened to a considerable depth by sprinkling before a pavement is laid. If the construction is on a dry clay soil, excessive saturation and minimum bearing values may occur before the concrete has attained normal strength."

Elastic properties of subgrade soil.—In order to test the elastic reaction of a subgrade under repeated deformations, a special apparatus was devised. As a result of the tests made with this device the following comment was made: "The results of these tests are not sufficiently complete to justify definite conclusions. Present indications, however, point to the probability that none of the clay soils exhibits sufficiently uniform elastic properties to justify an assumption of elastic subgrade supporting power for use in a design formula.

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[29] Highway research in Illinois, Transactions of the American Society of Civil Engineers, vol. 87, 1924, pp. 1181–1224.

It is certain that, for purposes of pavement design, it is not safe to place reliance on a bearing-power test involving only a single application of load."

Discussion of findings.—The soil investigations of the Bates Road were valuable in that they further emphasized the truth of similar observations which had previously been made. That tile drains are not effective in a clay soil and that the bearing power of a subgrade soil may be affected materially by its moisture content during the time of construction were well-known phenomena prior to these tests. On the contrary the tendency of a wetted subgrade to resist further saturation, while a subgrade originally dry will later absorb moisture in excess of that present in the wet subgrade which resists further saturation is a feature which seems to have been given little, if any, previous attention. The practice of wetting subgrades before the pavement is laid has always been emphasized, although the serious consequences of a greater saturation which might result if the subgrade was not wetted, has never been particularly stressed. This is a procedure which should receive attention. Added to this is the probability that a dry subgrade will absorb moisture from a freshly laid concrete pavement and, becoming saturated, may distort and cause hair cracking of the pavement before the final set has occurred. Another novel section of the report on the subgrades was the investigation of the elastic reaction of the soil under repeated loads. Of course this elasticity applies to clay rather than to sandy soils, but the possibility of a lag in returning to original position after deformation certainly indicates further the uncertainty of clay foundations for subgrades.

RESEMBLANCE BETWEEN THE PITTSBURG AND BATES ROAD SUBGRADE FINDINGS

The heavy adobe soil under the Pittsburg test road with the predominating moisture equivalent percentages varying from 40 to 50 was believed to have been made more impervious to water by unique methods used in constructing the pavement. The construction was accomplished by excavating and backfilling the subgrade in layers and rolling and sprinkling the successive layers. This appeared to retard the rate at which the soil later absorbed water. Whether the subgrade would permanently take up less water because of the treatment was believed to be a subject for further study.

The brown silt-loam of the Bates test road with a moisture equivalent percentage of approximately 30 was found when wetted during construction with a percentage of water normal to the summer months, to resist further saturation to a marked degree. It was believed that this added resistance might extend over a period of one or two years and thus be of value in providing a more substantial subgrade to the pavement during the time when it was developing a permanent set. On the contrary the pavement laid upon the dry subgrade, absorbed water rapidly later in the season to the point of saturation. This maximum water content exceeded that of the soil originally wetted with an amount of water normal to the summer months.

The Bates and Pittsburg findings were similar in that the subgrades wetted prior to construction seemed to resist further saturation. This might have been due to the puddling of the soils and a resulting rearrangement of the soil clusters or grains so as to produce a greater density, thereby making the soil more impervious to water. Or the seams resulting from the cracking of the adobe soils when shrinking might have been eliminated when the soil was rearranged. This would do away with the seams as water-bearing channels to accelerate the saturation of the soil. It may be that the Pittsburg test method of rolling and sprinkling the subgrade in layers further accomplished the rearrangement of the soil grains and clusters by physical or mechanical processes. A similar cause may have produced the results noted in the Bates Road test but there is nothing to indicate this in the reports. The important point to bring out, however, is the possibility that the characteristics of a subgrade soil may be improved by physical or mechanical processes. As to this possibility there is further evidence in the experience of former city engineer, Burwell Bantz, of Chehalis, Wash. Mr. Bantz states that he found that sidewalks laid on heavy clay soils cracked badly. To overcome this he excavated the top layer of the clay subgrade, backfilled, and tamped it into place and sprinkled. The cracking of the sidewalks was found to be materially reduced.

SOIL BEHAVIOR IN PRESENCE OF MOISTURE ALTERED BY CHEMICAL PROCESSES

That the behavior of a soil in the presence of moisture may also be altered by natural chemical processes is indicated by the following quotations:

"The physical condition of the soil and particularly its permeability to water is largely influenced by the character of the bases that are combined with the soil. When the alkaline bases, sodium and postassium, predominate the soil is deflocculated and impermeable. When the earthy bases, calcium and magnesium, are in excess the soil is flocculated and permeable.

"When saline soils are leached to reduce the concentration of the soil solution it is often found that they become impermeable to water. This condition is due to the effect of the alkaline bases combined with the soil which causes deflocculation to take place when the salts of the strong acids, sulphate and chlorin, are removed from the solution."[30]

And "according to experiments of Gedroiz, the enhanced swelling of the soil and dispersion of the colloidal material take place after the excess of soluble salts is removed, the presence of considerable quantities of the sodium salts tending to keep the colloidal material in a flocculated condition."[31]

It seems apparent, therefore, that bad subgrade soils may be transformed into good subgrade soils by natural physical or chemical phenomena. These natural phenomena which produce a permanent transformation of the soil occur over long periods of time. The Bates and Pittsburg test results indicate that a temporary improvement in the subgrade soils may have been accomplished rapidly by artificial means. The improvement of the character of bad subgrades by accelerated artificial chemical processes provides a fertile field for further investigation. If the active elements in a heavy clay could be rendered inert by an inexpensive chemical means it would revolutionize our present methods of pavement design.

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[30] The movement of water in irrigated soils, by Carl S. Scofield, Journal of Agricultural Research, vol. 27, No. 9, Mar. 1, 1924.

[31] Recent investigations of soil colloids, by Philip L. Gile. Proceedings of the American Society of Civil Engineers, vol. 51, No. 5, May, 1925.

BUREAU OF PUBLIC ROADS LABORATORY EXPERIMENTS AT THE ARLINGTON EXPERIMENTAL FARM

Coincident with this work done by other agencies the Bureau of Public Roads has been making investigations at the Arlington (Va.) Experimental Farm to determine the behavior of subgrade soils when acted upon by an external force and also to ascertain the characteristics of soil which determine the degree to which it will be affected by an external force or climatic conditions.

The most important of the investigations of the behavior of soils when acted on by external forces are described briefly in a recent paper published in the Transactions of the American Society of Civil Engineers.[32] This paper is quoted as follows:

"*Influence of the bearing value of the subgrade.*—The intensity of pressure on a road subgrade is governed by a number of factors. The wheel load is carried directly by the wearing surface which, in turn, transmits the pressure to the underlying subgrade. If the wearing surface is of a rigid type, as concrete, capable of resisting bending, the subgrade pressures are distributed over a comparatively broad area adjacent to the wheel, and the highest intensity of pressure is comparatively low. On the other hand, if the surfacing material is of a flexible type, incapable of resisting bending, the area of distribution of subgrade pressures will be quite restricted and the intensity of pressure under the wheel will be high. These statements are illustrated by curves of pressure distribution obtained under concrete and with broken-stone surfaces of equal thickness, both subjected to a concentrated load. (fig. 5.) The pressures were measured by soil pressure cells, described elsewhere.[33]

"The degree of yielding of the soil also affects the pressure distribution. Thus, a solid rock subgrade would have high and restricted pressures under the wheel load, whereas a soft soil would have low pressure intensities, well spread out, provided, of course, the overlying slab was of such a nature as to resist bending. The flexibility of the slab likewise controls the pressure intensity and distribution. Rigid slabs, such as concrete, can deflect little before a crack is produced; other types of pavements of less rigid character can be deformed more, without the accompanying incipient failure. The subgrade material, therefore, has to develop its resistance to the overlying load within safe limits of deformation of the pavement. It is quite important that the bearing value be obtained corresponding to a known amount of deformation of the subgrade. This amount will be less with rigid slabs than with nonrigid slabs."

The most important implication made by these data is the necessity for experimental studies to determine the bearing power of the various types of soils from gravel, through sand and silt to clay. These tests could possibly be made by gradually applied repeated loads (not involving impact) through concrete slabs to the various soil types each with degrees of moisture varying from the hygroscopic coefficient through the moisture equivalent percentage, to complete saturation. The critical load in the soil type would be determined when the soil fails to resume its original position (within reasonable limits) after deformation, while with further repetitions of the same load the deformation increases and the soil pushes out from under the slab and exemplifies the action of an unstable subgrade. By correlating the force of impact and statically applied loads it might be possible to determine a constant which might be used as a multiple of the observed critical load determined statically and the result would represent the limiting safe pressure which could be sustained by any soil type under the varying moisture conditions. Then by establishing the limiting value for the moisture content which should be permitted in a subgrade it should be possible to establish the greatest allowable pressure for any soil type as classified by the United States Bureau of Soils. Using a high factor of safety probably from 4 to 10 because of the variable character of soils, it might then be possible to arrive at the proper thickness of pavement of the various classes of broken stone, bituminous concrete and cement concrete, that it would be necessary to use in order to distribute the load over a large enough area to maintain the allowable pressure in pounds per square inch within the observed allowable pressure determined for the particular soil type after being divided by a high factor of safety.

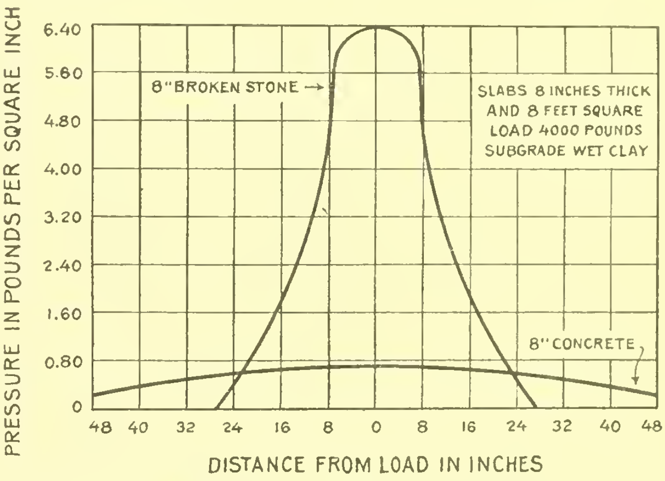

FIG. 5.—Pressure distribution to the foundation through broken stone and concrete slabs

"*Influence of bearing area on bearing value of soils.*—Another consideration in determining the bearing value of subgrades is that due to the different area of support offered by the subgrade, depending on the rigidity of the road slab. The bearing value of a soil, for a given penetration of the bearing area, depends on the magnitude of that area. It was with these considerations in mind that a series of tests was conducted to determine the influence of the size of areas on the bearing value of soils for definite penetrations of those bearing areas into the soil. These investigations were conducted, using bearing blocks of from several square inches up to 9 square feet in area, and with different types of subgrade materials, such as plastic clay and sand. Although there is some variation in the law for the different kinds of materials, the accompanying curve (fig. 6) is, in general, indicative of results obtained. It will be noted that when small bearing areas are used, the intensity of pressure required to produce a penetration of 0.1 inch far exceeds that for large-sized blocks. This is readily explained by the fact that when the large bearing areas are used a greater thickness of the soil is compressed, which contributes more toward the movement of the block than in the case of the small bearing area. It may not always be possible to make bearing value determinations by the use of large-size blocks. If smaller bearing blocks are used, it will be necessary to obtain the bearing value of the soil at a much less penetration, in order to obtain values corresponding to a definite penetration of the large bearing block. The relation between the depths of penetration of different sized bearing blocks for like intensities of pressure has been established by a series of tests using various bearing areas."[34] In Figure 7 is shown the results of this investigation, giving the relative penetrations of the areas under equal intensities of load. The experimental results for a constant unit load equal to that which causes a penetration of 0.1 inch over the area of 9 square feet seem to be quite accurately expressed by the relation:

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[32] Researches on the structural design of highways by the United States Bureau of Public Roads, by A. T. Goldbeck. Transactions of the American Society of Civil Engineers, vol. 88. p. 264 (1925).

[33] The distribution of pressure through earth fills, by A. T. Goldbeck, Proc. A. S. T. M., 1917.

$$\text{Area (square feet)} = 900 \times [\text{penetration (inches)}]^2$$

From this equation, it follows that:

$$\frac{\text{Penetration } (a)}{\text{Penetration } (A)} = \sqrt{\frac{a}{A}}$$

in which, a and A are the areas of the respective bearing blocks. Other experimenters have confirmed this relation.[35]

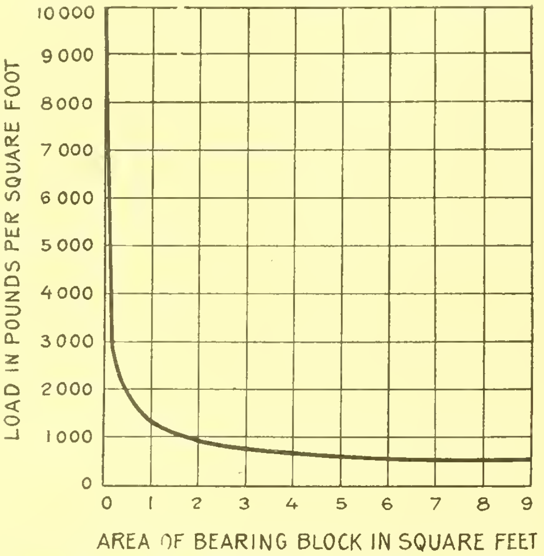

Fig. 6.—Relation of the unit load to the area of the soil bearing for a penetration of 0.1 of an inch

In regard to these bearing value tests, some very pertinent observations may be made. If to produce a penetration of 0.1 of an inch with a bearing block 3 square feet in area it took a load of 800 pounds per square foot and with a bearing block of 2 square feet area 900 pounds were necessary, it would seem that the practice of reducing the size of the slabs in concrete pavements by transverse and longitudinal joints would reduce the stress in the subgrade. However, it is not believed that this is the case because the greatest stress in a pavement occurs at the corner of the slab and the amount of this stress would not seem to be diminished by reducing the size of the slab. It must also be remembered that these tests apply to a uniformly distributed load over the area and not a concentrated load or a blow delivered by impact. Further, that the limit of the comparative deformations in each case for the several loads was 0.1 of an inch. The tests also were confined to observations made to determine the relative penetrations caused by the same unit of load. The relative loads necessary to produce a common penetration were not studied. Mr. Goldbeck points out that it is possible that this relationship might change with progressively increasing deformations of the subgrade beyond the so-called elastic limit and specifically limits the results to the elastic limit of the soil and logically this limit would seem to be the extreme condition in the design of a subgrade. The facts brought out by this investigation apply especially to the design of footings for bridges. The investigation indicated that the current practice of assuming equal deformations of the foundation soil with equal intensity of pressure regardless of the total area of the footing is erroneous. On the other hand, in order to obtain an equal settlement of the footings, a method should be used which would vary the allowable pressure with the area of the foundations.

IMPROVEMENT OF SUBGRADES BY ADMIXTURES

To the question as to whether it is possible to convert a plastic type of subgrade into a more desirable nonplastic type through some system of curative treatment, the Bureau of Public Roads answers by giving the results of experiments made in its laboratory. Figures 8 and 9 show these results; and it is evident from Figure 8 that additions of 5 per cent of lime and Portland cement decrease the volumetric change due to variations in moisture content and increase the bearing power up to the limit of the capillary moisture capacity of the soil. The use of both lime and Portland cement is very expensive, and to determine the possibility of reducing the volume change of bad subgrades with a cheaper admixture, comparative tests were made on medium sand and Portland cement, the results of which are shown in Figure 9. It appears from these results that medium sand is more effective than Portland cement in reducing the volume change in bad subgrades. The difficulty, from a practical standpoint, of breaking up the clumps in a clay subgrade and uniformly mixing the sand or other admixture raises a serious objection to this method of treatment. It

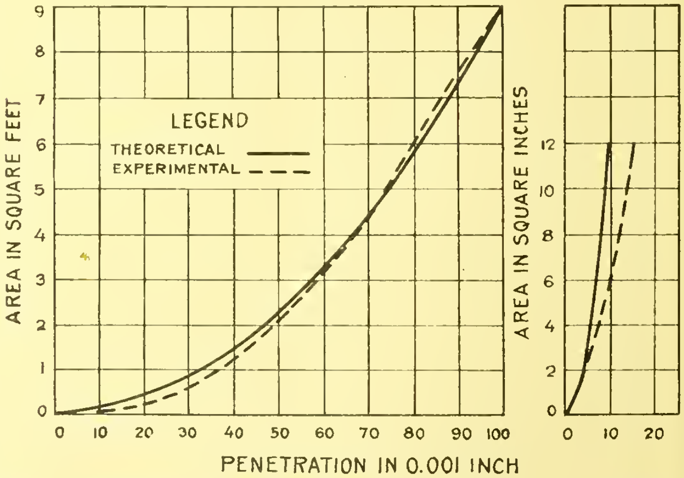

Fig. 7.—Relation of the penetration of the soil to the bearing area of the slab for equal unit loads

would seem therefore that a cheaper method would consist in laying a subbase of sand or gravel or broken stone over the bad subgrades.

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[34] Research on the structural design of highways by the U. S. Bureau of Public Roads, by A. T. Goldbeck, Trans. Amer. Soc. Civ. Eng., vol. 88, p. 274 (1925).

[35] See article in Le Genie Civil, May 26, 1923, by Bijls, of Belgium.

Another series of experiments, also by the Bureau of Public Roads, at Arlington has involved the testing of concrete slabs laid on specially prepared subgrades under the simulated impact of a motor truck wheel. Of the six slabs, exactly alike in dimensions and mix, four were laid on subgrades treated with admixtures of Portland cement. Table 2 gives the results of the tests.

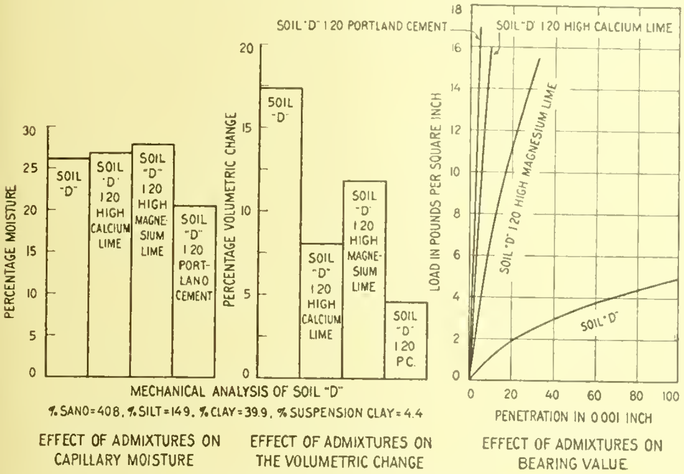

Fig. 8.—Effect of admixtures on the capillary moisture, volumetric change, and bearing value of a subgrade soil

Table 2.—*The resistance to impact of six similar slabs of concrete pavement laid on similar subgrade soils treated in various ways*

Slab No.	Admixture of Portland cement	Height of drop of motor truck wheel	Resistance of slab at failure
		Inches	*Pounds*
1	None	1.34	12,660
2	None	1.09	11,330
3	1 part Portland cement to 14 parts soil	.91	20,650
4	1 part Portland cement to 14 parts soil	.92	19,060
5	1 part Portland cement to 28 parts soil	1.04	18,250
6	1 part Portland cement to 28 parts soil	1.06	18,000

The weight used was equivalent to the load on the rear wheel of a 5-ton truck. The sprung weight was 6,000 pounds and the unsprung 1,980 pounds.

Slabs 1 and 2 were laid on the untreated natural subgrade prepared by excavating to a depth of 1 foot, backfilling in thin layers and tamping. The subgrades under slabs 3, 4, 5, and 6 were dug up in the same way and backfilled after being mixed with the desired proportions of Portland cement. The subgrades under slabs 4 and 6 differ from those under slabs 3 and 5 in that the former were excavated again, reshaped and retamped just prior to the placing of the concrete in order to provide conditions similar to those obtaining in actual construction.

The report of the investigation concludes:

"1. That the admixture of Portland cement with a subgrade soil of low supporting value will result in an improved supporting value.

"2. That relatively greater benefit results from the first few per cent of cement added than from subsequent equal amounts.

"3. That cutting up of this prepared subgrade after it has set up does not materially damage it

"4. Examination of some of this material which had been in place for two years indicates that its physical character is permanently changed. It seems to lose all plasticity and become very firm and granular, even in the presence of excessive amounts of water."

THE PROPER DEPTH OF SUBBASES

The proper depth of subbases under known conditions of subsoil, climate, traffic, and type of pavement is yet to be determined. In a general way tests made by the Bureau of Public Roads indicate that the intensity of pressure on a soft subgrade caused by concentrated loads on the surface decreases as the depth of the surfacing (including the subbase) increases. At this point it may be well to bring out the necessity for detailed tests on sand, crushed rock, or gravel of various sizes and grading used as a subbase under concrete and bituminous pavements to determine the intensity of pressure transmitted to the subgrade by a concentrated load at the surface of the pavement. The study would involve various depths of pavement and subbase and would necessarily be made on the various soil types such as sand, silt, and clay because the intensity of pressure at the subgrade with the same applied load at the surface of the pavement might vary with the elasticity of the soil. The loads would be gradually applied without any impact and later a constant might be determined which, multiplied by the static load, would make allowance for impact. This test in conjunction with similar tests previously mentioned to determine the maximum allowable pressures for various types of subgrade soil, should provide information which would do much to clear up the uncertainty which at present surrounds the subject of pavement design and throw light on the proper dimensions of subbase and surfacing to use under known climatic, subgrade, and traffic conditions.

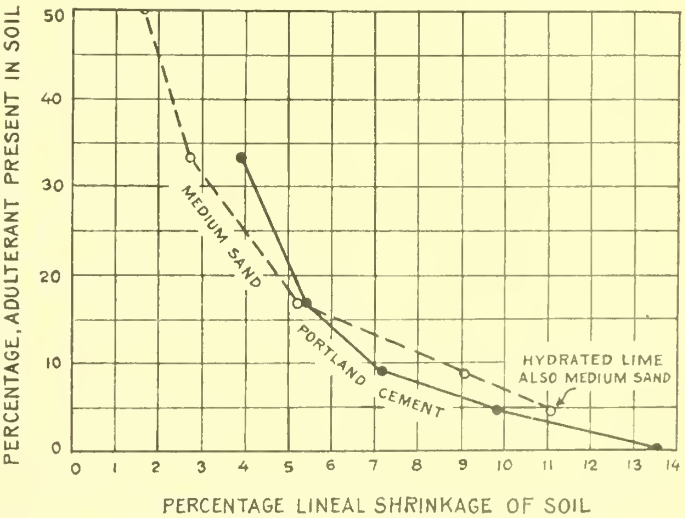

Fig. 9.—Effect of various admixtures upon the shrinkage of a sample of covo clay subsoil

As to the characteristics of the subgrade[36] which indicates a need for the construction of a subbase, the study of the Columbia Pike experimental road constructed near Arlington, Va., by the Bureau of Public Roads, has led to the conclusion that "subgrades that show as much as 10 per cent volume change, by laboratory test on an entire sample including coarse material should be covered with a layer of coarse granular material, and a pavement laid on a subgrade of this character should have a longitudinal joint at the center." The results of the laboratory tests for volume change in this case were corrected to represent the volume change which would have occurred with coarse material present and the sample wetted to capillary saturation. The limits determined in the Pacific Northwest[37] of 14 per cent volumetric shrinkage and 5 per cent lineal shrinkage were established without coarse material and at a moisture content equal to the moisture equivalent percentage. As nearly as can be estimated the upper limit of 10 per cent volumetric shrinkage with coarse material included corresponds to a volumetric shrinkage of from 17.4 to 26.3 per cent as found in the Pacific Northwest. Using the lower figure in order to be conservative, the comparison of the data from the two investigations follows:

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[36] Reinforcing and the subgrade as factors in the design of concrete pavements, by J. T. Pauls, Public Roads, vol. 5, No. 8, October, 1924.

	Limiting shrinkage determined in Columbia Pike experiments	Limiting shrinkage determined in the Pacific Northwest
	Per cent	*Per cent*
Volumetric	17.4	14
Lineal	6.3	5

RIO VISTA, CALIF., TESTS ON SUBGRADE ADMIXTURES

In connection with field tests made by incorporating admixtures in the subgrade soil to reduce the volumetric change with variations in moisture content, probably the most recent record of results is that made by C. L. McKesson, materials and research engineer of the California Highway Commission, who has given a great deal of study to the subject of subgrades. Mr. McKesson was in charge of the field subgrade soil tests made by the Bureau of Public Roads in the Pacific Northwest. Excerpts from a memorandum of his made on October 18, 1924, and a large portion of which has never been published before are given here.[38]

"In November, 1921, experimental subgrade treatment was begun on the Rio Vista lateral between Denverton and Rio Vista.

"The treatment consisted of loosening and pulverizing the soil to a depth of 6 and 12 inches, after which various adulterants were mixed with the pulverized soil.

"Eleven 500-foot sections and one 380-foot section were treated as follows:

"Section 1. Station 177+50 to 182+50, 1 to 10, cement mixture, 12-inch depth.

"Section 2. Station 182+50 to 187+50, 1 to 20, cement mixture, 12-inch depth.

"Section 3. Station 194+00 to 199+00, 1 to 10, cement mixture, 6-inch depth.

"Section 4. Station 212+00 to 217+00, 1 to 20, cement mixture, 6-inch depth.

"Section 5. Station 217+00 to 222+00, 1 to 20, hydrated lime, 12-inch depth.

"Section a. Station 248+50 to 253+50, 1 to 10, cement mixture, 6-inch depth.

"Section b. Station 253+50 to 258+50, 1 to 20, cement mixture, 6-inch depth.

"Section c. Station 258+50 to 263+50, 1 to 10, cement mixture, 12-inch depth.

"Section d. Station 263+50 to 267+50, 1 to 20, cement mixture, 12-inch depth.

"Section 6. Station 268+00 to 273+00, 1 to 20, limestone dust, 12-inch depth.

"Section 7. Station 273+00 to 278+00, no foreign substance.

"Section 8. Station 278+00 to 283+00, 60 per cent asphaltic oil, 12-inch depth.

"A detailed description of methods used in preparation of the various sections is to be found in the biennial report of the highway commission 1921–22, pages 61 to 64. * * *

"The subsoil on sections a, b, c, and d was treated in November, 1921, and the remainder in the summer of 1922. Pavement was constructed in the summer of 1922.

"The subsoil under the various sections is adobe and silty clay soil. There is no record in the laboratory of any analyses of the soil having been made or of any laboratory experimental work in connection with this test. In the description of the work it is stated that 'it was necessary to select segregated sections as there was no stretch of road which would permit of continuous section.' It is reasonable to assume, therefore, that an effort was made to select sections where subsoil conditions were as nearly identical as it was possible to secure. There is, however, a considerable variation in the quality of the subsoil on the various sections chosen. * * *

"A 4-inch gravel sub-base was placed on all heavy soil subgrade on the project except on the experimental sections and it is therefore possible to compare this more or less standard method of subgrade treatment with the various admixture treatments.

"The attached prints (fig. 10) show the condition of the pavement, station 240 to 285, on April 2, 1924, and station 172+50 to 227+00, on September 15, 1924.

"The relative efficiency of the various methods of treatment based on the present condition of the pavement is as follows:

"*Four-inch gravel subbase on untreated subsoil* is found to be very efficient. All sections in good condition. No longitudinal cracks and transverse cracks are 40 to 100 feet apart.

"*Six-inch Portland cement* (*secs. a, b, 3, and 4*).—Where subsoil is similar there is little or no difference apparent between sections having 1:10 and 1:20 admixtures. Failures consist of transverse cracks averaging 40 feet apart.

"*Twelve-inch asphaltic oil* (*5 gallons per square yard, sec. 8*).—This section has transverse cracks averaging 25 feet apart. There is little difference in condition between this section and the 12-inch Portland cement sections.

"*Twelve-inch hydrated lime* (*sec. 5*).—This section has transverse cracks 10 to 60 feet apart and several short irregular longitudinal cracks. Some surface checking was also noted.

"*Twelve-inch Portland cement sections.*—Sections 1 and c with 1:10 admixture are in slightly better condition than sections 2 and d having 1:20 admixture. Transverse cracks on average about 25 feet on sections 2 and d. Sections 1 and 2 have a number of longitudinal cracks. The average condition of these sections is slightly better than the plain, untreated section 7 but not enough to justify any expenditure.

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[37] Practical tests for subgrade soils, by A. C. Rose, PUBLIC ROADS, vol. 5, No. 6, August, 1924.

[38] See also Fourth Biennial Report of the California Highway Commission, Nov. 1, 1924, p. 77.

"*Plain 12-inch.*—This section was plowed up and soil pulverized as for admixture treatments. It was then rerolled without the addition of any adulterant. Pavement on this section is in a little worse condition than on 12-inch 1:20 cement section but the soil is apparently heavier than an any other section of the experimental work. A comparison of this section with gravel subbase sections shows clearly the benefit to be obtained from the use of a gravel subbase.

"*Twelve-inch limestone dust* (*sec. 6*).—This section is in the worst condition of all. The pavement is broken up into narrow strips by the many transverse cracks and in some places short longitudinal cracks are numerous. Had it not been for transverse reinforcement this section would probably be a total failure.

"When this experimental work was started in 1921, Mr. A. C. Rose, who had for 18 months previously been making soil studies in the United States Bureau of Public Roads district 1 under my immediate supervision, called to my attention the press reports on Solano-53-B. At my suggestion he immediately started a series of tests to determine the probable effect of the proposed adulterants upon what we believed to be the two most important characteristics of a soil, namely, shrinkage and moisture retaining capacity as indicated by its moisture equivalent. In addition to the use of lime, cement and limestone as adulterants, we included specimens adulterated with fine, medium, and coarse sand believing that these adulterants would be found equally efficient. The soil selected for the experiment was Cove clay which corresponds very closely with the soil on Sol.-53-B. It contained 44.9 per cent clay and 40.6 per cent silt against 41.6 per cent clay and 37.2 per cent silt in a typical sample from the Solano project.

"The results of the tests made by Mr. Rose are shown in Table 3.

TABLE 3.—*Showing effect of adulterants on shrinkage of Cove clay subsoil*

Kind of mixture	Moisture equivalent; average of 2 runs	Lineal shrinkage; average of 2 runs	Volumetric shrinkage (computed)
	Per cent	*Per cent*	*Per cent*
1 Portland cement to 2 soil	31.4	3.90	12.8
1 Portland cement to 5 soil	36.7	5.40	15.2
1 Portland cement to 10 soil	42.4	7.20	20.2
1 Portland cement to 20 soil	43.3	9.84	26.8
1 hydrated lime to 20 soil	47.3	11.10	29.8
1 pulverized limestone to 20 soil	41.4	13.82	36.0
1 medium sand to 1 soil [1]	20.1	1.64	5.0
1 medium sand to 2 soil	25.7	2.73	8.0
1 medium sand to 5 soil	35.4	5.30	15.0
1 medium sand to 10 soil	38.1	9.08	24.9
1 medium sand to 20 soil	39.4	11.10	29.9
1 coarse sand to 2 soil [2]	25.9	5.20	15.0
1 fine sand to 2 soil [3]	27.5	7.32	20.5
All soil	40.5	13.46	35.2

[1] Medium sand: Passing 10 mesh and retained on 20 mesh, 33 per cent; passing 20 mesh and retained on 40 mesh, 67 per cent.

[2] Coarse sand: Passing ¼ inch and retained on 10 mesh, 33 per cent; passing 20 mesh and retained on 40 mesh, 67 per cent.

[3] Fine sand: Passing 40 mesh and retained on 200 mesh, 100 per cent.

"The results were summed up as follows:

"1. Portland cement and ordinary sand of medium coarseness were found to be *equally effective* in lowering the moisture equivalent and shrinkage factors of a soil. A 1 to 10 or 1 to 20 mixture of either cement or sand with clay soil was found to reduce the shrinkage factor only a few points and the soil after such treatment was not suitable for use in subgrade for pavement. Such admixtures were therefore reported to be useless. In order to lower shrinkage to a point of approximate safety (5 per cent lineal shrinkage) it was found necessary to use one part of cement to two parts of soil. Such a treatment would of course be extremely costly. Sand used in the same proportion (1 to 2) changed the soil from a clay to a clay loam and reduced the shrinkage somewhat more than did the Portland cement.

"2. Hydrated lime in 1 to 20 proportion was found to be practically useless, the lineal shrinkage being reduced only 2 per cent (13 to 11 per cent).

"3. Pulverized limestone was found to be positively detrimental in that it increased both shrinkage and moisture equivalent.

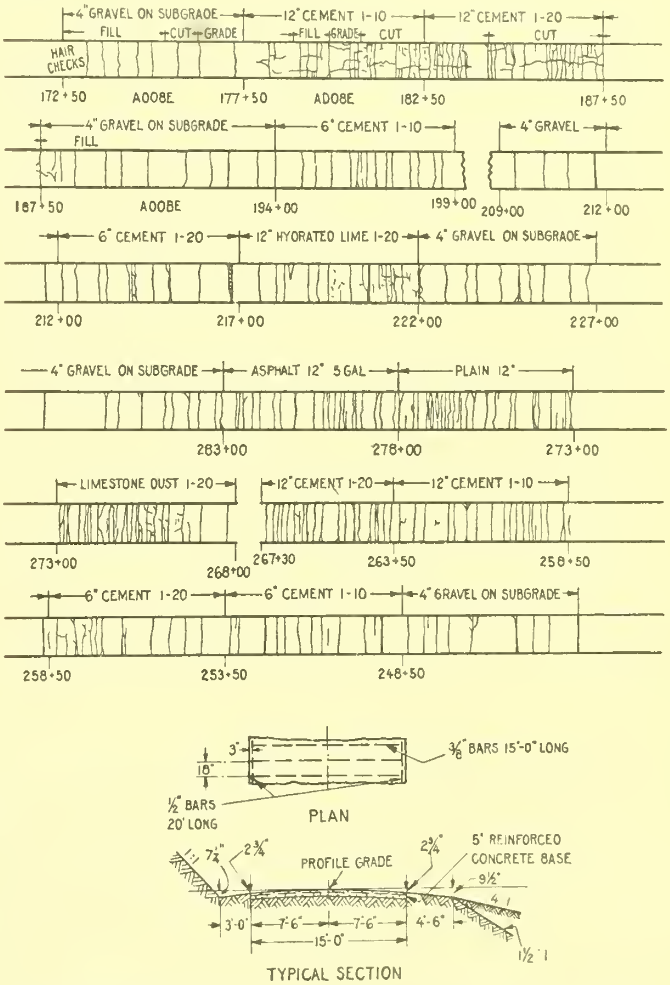

FIG. 10.—Condition chart of the concrete pavement on the Denverton-Rio Vista (Calif.) experimental road, three years after the various subgrade treatments were begun and two years after the pavement was completed

"The conclusions from the laboratory experiments made two years ago are entirely verified by the present condition of the several experimental sections.

"In the report of his tests, Mr. Rose stated the conclusion which we had reached as a result of this and many other investigations. It is so pertinent to the subject now under consideration that it may well be quoted at this time.

"'Adulterating the subgrade with a lime compound or sand to a depth of 1 foot reduces shrinkage in the portion adulterated, but the subsoil for a considerable depth beneath this treated layer would continue to swell and shrink and displacement of the upper layer and of the pavement would doubtless continue although to a less degree. The efficiency of this method of subgrade treatment is doubted. It is believed that a better treatment of heavy soil subgrade is:

"'1. To use a sand cushion to act as an equilibrant to run into irregularities of the subgrade which is deformed by shrinkage or swell of the soil and thus to maintain a uniform surface in contact with the base of the pavement,[39] or

"'2. To establish an unchanging moisture content in the immediate subgrade.'

"The mixed sand and gravel treatment and a special side ditch design was adopted in 1921 on a 4-mile stretch of pavement [40] constructed on this same Cove clay and this spring after two and one-half years the pavement without reinforcement was entirely free from longitudinal cracks and transverse cracks were infrequent. (Expansion joints were used at 30-foot intervals.)

"The results of the Rio Vista lateral experiment appear to justify three conclusions:

"1. That the soil adulteration with cement or lime compounds is not an efficient or economical method of securing stability in heavy soils.

"2. That the suitability of the soil for subgrade purposes, or the merits of various methods of soil treatment can be determined by relatively simple laboratory tests and that expensive field tests can, in some cases at least, be avoided by first resorting to a properly conducted laboratory investigation.

"3. That a sand or gravel layer is an efficient and economical method of minimizing damage to pavement resulting from swelling or shrinkage of the subsoil."

The costs per lineal foot of roadway for each of the methods of treatment as shown by the cost records of Mr. McKesson follow:

Section No.	Treatment of subgrade		Cost per lineal foot
1	1 to 10 cement mixture	12-inch depth	$2.20
2	1 to 20 cement mixture	do	1.275
3	1 to 10 cement mixture	6-inch depth	1.265
4	1 to 20 cement mixture	do	.752
5	1 to 20 hydrated lime mixture	12-inch depth	.921
6	1 to 20 limestone dust mixture	do	.817
7	No foreign substance		.189
8	60 per cent asphaltic oil	12-inch depth	.957

The cost of the 4-inch gravel sub-base is not available but Mr. McKesson estimates this at $4 per cubic yard in place or $0.74 per lineal foot of pavement.

It should be remembered, however, that these Rio Vista experiments were carried on under unique conditions. The adobe soil and the hot, desiccating winds provided a dry, parched subgrade which made it essential to wet the soil thoroughly before the pavement was laid. If there was any difference in the amount of wetting of the various sections, it is possible that the amount of cracking of the pavement might have been affected considerably. It is also interesting to compare these results with the experiments of the Bureau of Public Roads at the Arlington Farm. On the Columbia Pike Road the section of subgrade treated with Portland cement in the proportion of 1 part of cement to 28 parts of soil to a depth of 6 inches, showed up remarkably well. This section had no longitudinal cracks as compared with the two adjacent untreated sections, where the pavement had cracked badly. The soil was a clay. Other experiments of the Bureau of Public Roads which are described elsewhere in this article indicate that the bearing power of a subgrade soil is materially increased by the addition of Portland cement as an admixture. It would seem, therefore, that the results of all these experiments with admixtures are inconclusive. Sufficient work has not been done to determine the practicability or impracticability of using Portland cement or hydrated lime to improve the quality of subgrade soils.

SEMIGRAVEL, TOPSOIL, AND SAND-CLAY ROADS

Dr. C. M. Strahan, who has made a comprehensive study of semigravel, topsoil and sand-clay roads,[41] states that: "The excellent topsoils are most largely found on the ridges of northeast Georgia where they lie in many deposits weathered in place from limited outcrops of porphyritic granites and coarse-grained sandstones. Here also many of the semigravel types are found consisting of partially decomposed granite soils. Other semigravels are found along the margin of an old geologic sea beach which enters the State on the east in Lincoln County and swings in a wide southwesterly arc to the Alabama line." And further on in the bulletin he states: "Southwest Georgia, notably adjoining the Flint River, abounds in sand-clay soils derived from Cretacious rocks and carrying from 5 to 25 per cent of a nodular iron-silica gravel. These deposits are liberally distributed and from them have been built a large mileage of remarkably efficient roads." These roads, as described by the Georgia bulletin, provide a low-grade but efficient and inexpensive surfacing material.

From an investigation of the condition of existing roads under different classes of traffic the author has evaluated the extreme limits by percentages of the coarse material, sand, silt, and clay for the three types of surfacing called class A (hard), class B (medium), and class C (soft). It is evident from an examination of these data that the methods outlined could be used elsewhere in the country as well as in the Southern States. It is also evident that the same low-grade types of surfacing are used elsewhere although they are not known by the same name, but the thorough study of the qualities of the material given by the University of Georgia has not been made elsewhere. For example in central Oregon, near the town of Burns, suitable standard gravel surfacing was not found for a Federal-aid project. This road extended over an old geological lake bed with coarse sand and pea gravel within three feet of the surface soil which was composed of volcanic ash. The latter was stripped from the semigravel deposit which was used to build an excellent surfacing for the moderate traffic on the project. Undoubtedly there are innumerable instances where the same treatments could be and have been used in other sections of the country.

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[39] It has been suggested that besides the vertical distortion of the subgrade due to shrinkage there are tensile stresses developed by the horizontal shrinkage of the soil. There is adhesion between the base of the pavement and the subgrade. It is possible that the granular layer destroys this adhesion by providing a sliding plane, which prevents high stresses being developed in the pavement due to the horizontal shrinkage of the subgrade soil.

[40] Oregon Federal-aid project No. 51.

[41] Research work on semigravel, topsoil and sand-clay, and other road materials in Georgia, Bulletin of the University of Georgia, vol. 22, No. 59, Serial No. 326, June 1922.

Sand-clay and semigravel roads studied, PUBLIC ROADS, vol. 5, No. 8, October 1924 p. 16.

The Georgia bulletin calls attention to the fact that, "Knowing the variable supporting power of ordinary soils under wet conditions, if selected road soils applied to the graded road bed can give them a uniform stability such soils offer a valuable treatment of the subgrade preceding the laying of pavements."[42] This is unquestionably true. On the other hand there is a possibility that the graded classification of topsoil, sand-clay, and semigravel surfacing material may be interpreted by some as a logical general method of differentiating between existing good and bad subgrade soils although Doctor Strahan has never advanced such an interpretation. The area shown on the trilinear chart in Figure 11 represents the extreme limits of the sand-clay, topsoil and semigravel classification. The clay fraction has been divided by two and included with the silt to make a comparison with the soil grain sizes of the Bureau of Soils. It will be observed that the area is only a small portion of the sand and sandy-loam classification of the Bureau of Soils. The coarse material present in the topsoil or similar material is not represented in the trilinear chart and need not be considered because of its obviously beneficial nature. It seems evident, therefore, that there are large areas of sand, sandy loams, loams and silt-loams which would make good subgrade material without treatment, but which are not included within the limits adopted for sand-clay, topsoil and semigravel roads. The classification would, therefore, seem to be of very restricted value for evaluating good and bad subgrades.

CAPILLARY MOISTURE AND ITS EFFECT ON SUBGRADE SOILS

During the past few years considerable attention has been given to the effect of capillary moisture upon the stability of highway subgrades. Free water is commonly distinguished from capillary water as the water in the soil which is not held by capillarity and obeys the laws of gravity. But tests conducted recently by the division of agricultural engineering of the Bureau of Public Roads tend to qualify this generally accepted definition differentiating between capillary and free water on the basis of gravity. The report of the tests, as published in a bulletin of the Department of Agriculture,[43] states: "From these data and as a result of tests by the writer and others, it can be said that a vertical soil column can take up by capillarity from a body of free water more water than it can hold against gravity, if the free water be removed from the bottom of the soil column; that is, if the vertical tube is filled with soil and the lower end placed in a vessel of water and allowed to stand for a month or longer and the water is then removed from the tank, a part of the moisture in the soil column will drain out. To repeat—a vertical soil column will take up by capillarity from a body of water more moisture than it can retain when the source of the water is removed." It would appear that the amount of capillary water in a subgrade where the source of the moisture is a wet soil would be less than when the source is a body of free water. The capillary moisture in a soil might, therefore, be reduced by lowering the ground water table considerably where it is not possible to eliminate it altogether. Often, however, it is neither possible nor practicable to lower or eliminate the ground water table.

As a result of the subgrade drainage experiments conducted by the Bureau of Public Roads at the Arlington Experimental Farm, "it is concluded that systems of drainage merely remove the free water in the subgrade, but can not remove capillary moisture. * * *."[44]

Charles M. Upham, director of the highway research board, and H. F. Janda, the assistant director, conclude:[45]

"1. That drain tile are of no direct value in removing or preventing the rise of purely capillary moisture.

"2. That the percentage of capillary moisture is reduced as the elevation of the subgrade above the water table is increased. Therefore, a tile drain might reduce the percentage of capillary moisture by lowering the water table."

Some interesting comments on the relation of capillary water to highway subgrades were made as early as 1921, by an irrigation engineer of the Bureau of Public Roads.[46]

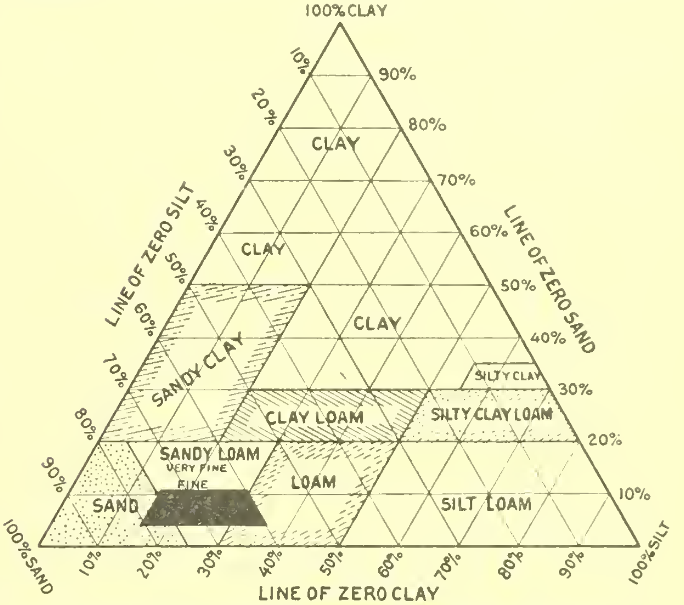

FIG. 11.—The black area represents the approximate limits of the sand-clay surfacing and also of the sand, silt, and clay fractions of the topsoil and semigravel classification

The concensus of opinion seems to be that it is not possible to remove capillary water from highway subgrades. Only the free water may be removed by practical methods.

EXPERIMENTS WITH ADMIXTURES FOR THE IMPROVEMENT OF EARTH ROADS

A number of experiments have been carried on recently by the Bureau of Public Roads and other agencies to determine whether the wearing qualities of earth roads may be improved by the addition to the soil of such admixtures as lime and Portland cement. None of the experiments has thus far developed information of direct value for the solution of the subgrade problem, a fact which is not surprising in view of the different purposes of the admixture in the two cases. In the first case the objects of admixture are to reduce the plasticity and increase the bearing power of the exposed surface soil and so make it less susceptible to rutting and capable of being used as a wearing surface throughout a greater portion of the year than would otherwise be possible. For this purpose the curative is mixed to comparatively shallow depths; and the plasticity of the surface is further reduced by unrestricted evaporation. In contrast to this, the admixtures used to improve the stability of a subgrade under a pavement must not only reduce the plasticity and thus increase the bearing power but they must also eliminate or minimize the volume changes of the soil with variations in moisture content or freezing. This volume change affects the soil to a considerable depth—at least a foot and probably more. In addition, the evaporation is restricted in this case by the pavement and condensation may occur. It would seem, therefore, inadvisable at this time to forecast beneficial or adverse results of subgrade admixture based upon the information at hand in regard to the effect of admixtures on earth wearing surfaces, or vice versa. However, the two problems are not totally dissimilar, and it may be well to record briefly the principal developments of the surface investigations, if for no other reason than the possibility that they may suggest to some one a new approach to a solution of the subgrade problem.

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[42] Research work on semigravel, topsoil and sand-clay, and other road materials in Georgia, Bulletin of the University of Georgia, vol. 22, No. 59, Serial No. 326, June, 1922, p. 5.

[43] Capillary movement of soil moisture, by W. W. McLaughlin, U. S. Dept. of Agric. Bul. No. 835, p. 62.

[44] Researches on the structural design of highways by the United States Bureau of Public Roads, by A. T. Goldbeck, Transactions of the Am. Soc. of Civil Eng., vol 88, p. 264, 1925.

[45] Removal of capillary moisture in highway subgrades, by Charles M. Upham and H. F. Janda, Engineering News-Record, col. 93, No. 23, Dec. 4, 1924, p. 912

[46] Capillary moisture and its effect on highway subgrades, by W. W. McLaughlin, PUBLIC ROADS, vol. 4, No. 1, May, 1921, p. 6.

The first of the field experiments of the Bureau of Public Roads, made in cooperation with the State Highway Department of Iowa, was built north of Chariton in Lucas County during the summer of 1924. The road consisted of 17 sections 400 feet long, treated with proportions of hydrated lime and Portland cement varying from 3 to 9 per cent. The engineer [47] in charge of the tests for the bureau reports, with respect to the effect of the treatment, as follows: "On September 1 (1924) we had the opportunity of studying the effect of the cement-treated sections. It was found that all colloidal properties had been destroyed. No plasticity whatever remained and in consequence of this fact the treated section showed no disposition whatever to make mud. To all intents and purposes it was as firm and mudless as any pavement, lacking only the characteristic firmness of the concrete slab. Even after we had cut into the surface a considerable distance no tendency whatever was manifested to stick as is the case in ordinary untreated gumbo when wet.

"One thing is certain, the cement does destroy colloidal conditions but whether or not a road so constructed will prove dustless is a matter for time to disclose."

On December 11, 1924, W. O. Price, of the Iowa State Highway Department, made an inspection of the project after a freezing spell to determine if there were any truth in the local reports that the experimental road was a failure. His report states: "On the experimental road we found several places where ruts had been cut down 4 or 5 inches deep with intermediate ridges. This was especially noticeable on the south two sections of the lime treatment, which are 3 and 5 per cent, and also on some of the other 3 per cent pieces. It was noticeable, however, that the frozen part was not so hard. It was either dryer and consequently had not frozen so hard or else it had thawed more rapidly. On practically all of the other sections even though they appeared rough on the surface one could drive a car at most any place without noticing the rough condition of the road, the surface of which was considerably dryer than the untreated sections. I believe that they could readily have been dragged at that time or perhaps an hour or two later in the afternoon whereas nothing could have been accomplished on the untreated road in that direction except possibly at some places along the edge where traffic had not gone while they were muddy. * * *

"To sum up, it appears that the treatment permits the working of the road earlier after a rain than the untreated road."

Another of the bureau cooperative experiments with the State of Iowa was near Glenwood in Mills County. The project consisted of 9 sections, each 1,000 feet in length and treated with 3, 5, 7, and 9 per cent of hydrated lime and Portland cement. The depth of the admixture was 6 inches. The project was built during September, 1924. On January 22, 1925, a Bureau of Public Roads engineer [48] reported:

"On the date of this inspection the ground was frozen to a considerable depth. The day was warm and the surface of the roadway had thawed to a depth of about 1 inch. The earth road where no treatment had been made was rutted, rough and quite slippery. It was quite difficult to turn a car out of the ruts.

"The difference in the condition of the 3 per cent road and the ordinary earth road was not noticeable. A marked improvement was quite noticeable on the remainder of the project. A considerable portion was fairly dry and but slightly rutted. A few places were somewhat slippery especially where the road surface was not fully exposed to the sun and wind. At this time there appeared to be no appreciable difference between the cement and lime treatments * * *.

"It is quite evident that a beneficial effect has been produced by the admixture treatment of cement and lime on the 5, 7, and 9 per cent sections especially during the winter months. Just how long * * * this treatment will be effective is quite problematical * * *."

The cooperative project, in which the bureau joined with the State Highway Department of South Dakota, was located on a road near Fort Pierre, S. Dak. The gumbo surface was treated in October, 1924. There was practically no rainfall during the following winter and on May 21, 1925, the bureau's inspecting engineer [49] reported as follows: "I found the several sections to be in substantially the same condition as untreated gumbo. The surface of all sections appeared to be substantially pure gumbo for about 1 inch in depth and below this was a mixture of more or less finely divided gumbo with cement or lime. The cement or lime is still in a dry powdered state and does not appear to have received any moisture since it was placed on the road."

Other experiments, practically completed by the bureau, at the Arlington Experimental Farm, indicate that hydrated lime and Portland cement admixtures will improve the wearing surface of earth roads. At least the results found to date are promising. Whether the cost of the treatment will be of sufficient value to warrant the expenditure or whether the beneficial results are lasting will require more extended observations. It is too early yet to formulate any definite conclusions.

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[47] W. L. Spoon, senior highway engineer.

[48] C. H. Semper, associate highway engineer.

[49] A. I. Ostrander, highway engineer.

THE MISSOURI EXPERIMENTS

The engineering experiment station of the University of Missouri has recently conducted a series of tests similar to those reported above.

With regard to these tests, Prof. E. J. McCaustland writes as follows:[50]

"At a somewhat later time selected stretches of clay roads, subject to heavy traffic, were treated with varying percentages of hydrated lime and these stretches have been kept under observation for a number of months. The winter season of 1924–25 was an extremely trying one on these test roads because of a heavy fall of sleet and ice early in December and a continuation through the winter of alternating periods of thawing and freezing. In every case, however, these test sections, all of which have been carrying traffic during the past 7 to 11 months, show to the most casual observer the distinct advantage of lime admixtures in preventing surface stickiness and the formation of deep ruts under traffic. In brief, it seems to have been established beyond question that the addition of hydrated lime in the proportion of 2 to 5 per cent dry weight to clay soils will materially reduce the tendency to form mud and prevent to a great degree the formation of ruts; there is reason to believe, however, that such treatment will not in any degree lessen the production of dust and in some cases the dust may be increased." The writer continues by pointing out that the maintenance costs are considerably reduced by the lime treatment since workers are able to begin work when the road is soft and workable immediately after a rain. There also appears to be no marked difference in the results caused by incorporating the admixture to different depths, and this is considered of importance since it affects the cost of treatment.

Professor McCaustland goes on to say that only negative results have been found in trying to determine the basic physical phenomena which are brought about by the lime admixture, and adds: "These facts are apparently established, however. The specific gravity of the soil is only slightly reduced by adding hydrated lime, the moisture holding capacity and moisture equivalent are reduced in heavy clays: The rate of evaporation is slightly retarded; but there is a more rapid capillary movement of the moisture after treatment; the voids in the soil became enlarged allowing an increased rate of percolation; and the bearing power at the higher moisture content is increased. We can not, however, at this time offer a scientific explanation of these phenomena."

The results of these experiments are interesting in so far as they suggest the possibility of improving the all-year condition of a large mileage of low-type roads throughout the country. It should be remembered, however, that the experiments are not conclusive of the ultimate benefit to be obtained by lime admixtures because of the short time during which the tests have been made—less than a year. It is possible that the further lime treatments made necessary by traffic may make the cure more serious than the disease because of the irritation of the nose, throat, and eyes by the lime dust during the dry seasons of the year. This is suggested as a possibility by Professor McCaustland when he writes: "Such treatment will not in any degree lessen the production of dust and in some cases the dust may be increased." The investigations are worthy contributions to professional literature although it is believed that final judgment as to the ultimate value of the lime admixture for improving the all-weather qualities of winter roads should be withheld pending further observations over a period of years or accelerated tests which shall take into consideration seasonal changes and additional treatments.

CONCLUSIONS

Relating to subgrade studies completed or in progress.—1. Generally speaking, all the data obtained in the United States indicate, the character of the clay remaining constant, that good subgrade soils have a low clay content by mechanical analysis and bad subgrade soils are characterized by a high percentage of clay. An illustration of an exception is the "rotten limestone" formation in the Black Waxy Belt of Texas, which, though high in clay is not a bad subgrade material. Pending further investigations, this finding should be accepted with reservations in this country and should not be applied to the soils of foreign countries without a thorough check against local conditions because there are well-known exceptions. An example of the latter are certain soils of the humid, tropical sections of Central and South America. With as much as 90 per cent of clay by mechanical analysis these soils are porous and not sticky. This is believed to be due to the oxidation of the colloids throughout the centuries in the warm, wet climate.

2. In general, the laboratory tests by the Bureau of Public Roads show that soils with a high percentage of clay have a high moisture equivalent percentage, a high capillary moisture capacity, a high volumetric shrinkage percentage, a low comparative bearing value, a high dye adsorption percentage, and require a long time for slaking.

3. Field methods for identifying good and bad subgrade soils have been devised by the Bureau of Public Roads. These involve the use of Bureau of Soils bulletins to locate the soil types and field tests for determining the moisture equivalent and lineal shrinkage percentages in order to evaluate the character of the subgrade soil.

4. The construction methods which may be adopted in bad subgrades, assuming the quality of the materials and the workmanship on the pavement to be uniform, are in general as follows:

(*a*) Use upland coarse grained soils for building fills over lowland clay soils.

(*b*) Use side ditches of special design.

(*c*) Use tile drains. In heavy clays these are generally useless except in uncommon cases such as open seams or pores in the soil, water-bearing strata, or hydrostatic pressure.

(*d*) Use a granular subbase such as sand, sand-clay, topsoil, stone, or gravel.

(*e*) Thicken the pavement.

(*f*) Add steel reinforcement.

5. The study of the California highway system by the Bureau of Public Roads showed that 70 per cent of the pavement failures occurred on adobe soils. The cracking of the pavements on adobe soils indicated a distortion of the subgrade due to varying moisture content and shrinkage.

6. The laboratory studies of the Bureau of Public Roads indicate that the moisture equivalent percentage of a subgrade soil is a critical percentage with respect

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[50] Dirt roads improved by lime mixed with surface soil, by E. J. McCaustland, dean, School of Engineering, University of Missouri. Extract from paper read before National Lime Association at Briarcliff Manor, New York, May 26-29, 1925, Engineering News-Record, vol. 94, No. 26, June 25, 1925.

to the bearing power of the soil. This does not mean that soils with a moisture content in excess of the moisture equivalent percentage will not support reasonable loads but that beyond the moisture equivalent percentage the bearing power of the soil falls off rapidly.

7. The field experiments of the Bureau of Public Roads in the Pacific Northwest indicate that a lineal shrinkage percentage of 5 per cent is, for that region, the maximum for a good subgrade soil. The Columbia Pike experiments of the bureau roughly checked this figure. The limit in the latter case was approximately 6 per cent.

8. Defining the term "stability ratio" as the actual moisture content percentage of the soil divided by its moisture equivalent percentage, the field investigations of the Bureau of Public Roads in the Pacific Northwest indicate that the bearing power of a soil is relatively low when the stability ratio is greater than unity. This does not apply to frozen soils. The bearing power of the soil by laboratory tests is relatively low also when the "moisture index" is greater than unity.

9. Clay soils, as compared with sands, show a relatively large heave when frozen according to field tests by Stephen Taber, State geologist of South Carolina.

10. Bureau of Public Roads laboratory tests show a greater percentage of water freezable in sands than in clays. This does not mean that the amount of the heave in sands due to freezing will be greater than in clays. On the contrary, it would seem that the greater heaving would occur in clay with capillary water present because of the greater total amount of water frozen.

11. In climates where freezing occurs, field observations indicate that for concrete pavement slabs of 60 feet or less in length:

(*a*) Cracks are infrequent in pavement laid on well-drained sand or porous subgrades.

(*b*) The more compact the subgrade soil, the more frequently cracks occur.

12. The permeability of soils or the porosity seems to increase with the size of the soil grains, the character of the grains remaining constant.

13. The best subgrades are found in fills and the worst in cuts.

14. The selection and redistribution of subgrade soils to secure a stable roadbed is a possible method of construction. The use of a granular subbase over the undisturbed subgrade soil is the more general practice.

15. A sand subbase prevents clay subgrade material from working up into the voids of a macadam road.

16. The Pittsburg and Bates Road soil findings indicate that the character of a subgrade soil may be improved by physical or mechanical processes.

17. The investigations of the Bureau of Public Roads indicate that the bearing value of a soil, for a given penetration of the bearing area, depends on the magnitude of the area. When small bearing areas are used, the intensity of pressure required to produce a penetration of 0.1 inch far exceeds that for large-sized blocks.

18. Sufficient work has not been done up to this time to determine the practicability or impracticability of using admixtures such as Portland cement or hydrated lime to improve the quality of subgrade soils.

19. Granular subbases seem to be more beneficial on bad subgrades than admixtures of Portland cement or hydrated lime.

20. Sand is as effective as Portland cement or hydrated lime when used as an adulterant to reduce the shrinkage of a clay soil.

21. Semigravel, topsoil, and sand-clay roads provide effective surfacing material in the Southeastern States. They could be and have been used elsewhere but under other names.

22. The hair cracks occurring during the curing period of concrete pavements laid on the loess soil of western Iowa were eliminated practically by a layer of tar paper on the subgrade.

23. Investigations made by the Bureau of Public Roads indicate that it may be possible to control the maximum moisture content of a subgrade to approximately the moisture equivalent percentage. Further studies are needed to confirm or disprove this.

24. It is not possible to remove capillary water from subgrades by drainage. Free water only may be removed by practical methods.

Relating to future subgrade soil studies.—Subgrade studies[51] which seem to be immediately necessary to accelerate the progress of subgrade soil investigations should involve probably not only the internal characteristics of different subgrade soils under varying physical and chemical conditions, but also the limiting external forces which may be applied to the subgrade soils under given conditions.

The specific studies which seem to be required immediately are:

1. A classification of soils by series and type names to conform as nearly as possible with the nomenclature and grading used by the United States Bureau of Soils in its soil survey bulletins.

2. The development of simple and decisive laboratory and field tests for subgrade soils.

3. The determination of the distribution of pressure to subgrades of various types through pavements and subbases of varying thicknesses and kind.

4. The determination of the maximum pressure per unit of area permissible, with various types and thicknesses of pavements and subbases, for different soil types with varying percentages of sand, silt, and clay.

5. The determination of the maximum moisture content of well-drained soils, by types and regions, throughout the year, and the maximum moisture content below which it is possible to control the water content of soils by types and regions.

6. The determination of the characteristics of soils, by types and regions, which make one more plastic than another and subject to a greater volume change with variations in moisture content, etc. If found that the active part of a soil is rendered inert over long periods of time by natural physical and chemical phenomena, investigations should follow to determine if similar results may be accomplished by accelerated artificial processes.

7. The determination of the maximum allowable shrinkage limits of soils wetted with comparable amounts of water (such as the moisture equivalent or capillary percentages) by soil types and regions. This would involve condition studies of existing pavements.

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[51] Digest of problems in the structural design of highways suggested by contact men in the various State highway departments, Proceedings of the Fourth Annual Meeting of the Highway Research Board, Division of Engineering and Industrial Research, National Research Council, 1925, p. 66.

8. The determination of the mechanical analysis, moisture equivalent and other test limits of those soils with varying degrees of permeability such that tile drains are either unnecessary, effectual, or worthless.

9. The discovery of the stages which accompany the phenomena of freezing of soils under existing pavements:

(*a*) Under continuous freezing conditions.

(*b*) Under conditions of alternate freezing and thawing.

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CORRECTION

In the article on colors and forms of traffic signals proposed by the sectional committee of the American Engineering Standards Committee which appeared in the August issue of Public Roads, under Rule 52 on page 136 the color temperature erroneously printed as 2.360° K should properly have been 2,360° K.

HIGHWAY RESEARCH BOARD ISSUES BULLETIN

THE Proceedings of the Fourth Annual Meeting of the Highway Research Board, recently issued, is a valuable contribution to scientific literature. It contains complete reports of the recent studies of the six active research committees of the board and is an extremely attractive bulletin.

The research committees of the board are organized as follows:

1. Committee on Economic Theory of Highway Improvement. *Chairman*, T. R. Agg, Iowa State College.
2. Committee on Structural Design of Roads. *Chairman*, A. T. Goldbeck, U. S. Bureau of Public Roads.
3. Committee on Character and Use of Road Materials. *Chairman*, H. S. Mattimore, Pennsylvania Highway Department.
4. Committee on Highway Traffic Analysis. *Chairman*, Geo. E. Hamlin, Connecticut Highway Commission.
5. Committee on Highway Finance. *Chairman*, H. R. Trumbower, U. S. Bureau of Public Roads.
6. Committee on Maintenance. *Chairman*, W. H. Root, Iowa Highway Commission.

The committees are composed of those men who have contributed to the scientific development of the highways and who are decidedly active in highway research.

The report of the Committee on the Economic Theory of Highway Improvement contains tables of the rolling plus air resistance for busses and automobiles with high-pressure cord tires and for trucks with pneumatic tires; data on the relation between the type of road and fuel consumption; highway transport costs; and the results of a study of tire wear. This report also contains a description of the studies now being carried on to determine the wind resistance of motor vehicles.

The report of the Committee on Structural Design of Roads contains reports on subgrade investigation; sand-clay road investigation; impact and static-load tests on road slabs; motor-truck impact tests; fatigue of concrete; theory of stresses in road slabs; stress measurement in road slabs; and skew-arch investigations. This report is well illustrated and contains much valuable information.

The Committee on Highway Finance report deals with several aspects of the financial questions so frequently encountered in a highway construction program. The report contains figures, charts, and tables that show such relations as total motor-vehicle revenues and total highway expenditures; gasoline tax rates in effect on July 1, 1924; highway expenditure and income; and other items that should be considered in the financial problem.

The report of the Committee on Highway Traffic Analysis contains interesting information on such topics as: (1) A study of the increase in motor vehicle registration to determine factors for forecasting future traffic; (2) the width of highway required to be constructed for the estimated life of the type of construction; (3) the economic value of eliminating bottle-neck points; and (4) highway transport surveys. A complete outline of highway transport surveys is given and should prove of value to all engineers interested in this subject as it indicates such items as the cost of the surveys, typical organizations and equipment required, and the kind of data to be taken.

The report of the Committee on Character and Use of Road Materials is extremely well illustrated and is indeed valuable information. The report contains studies of soundness tests for coarse aggregates; absorption of concrete in water as affected by the aggregates; effect of grading of the mineral aggregates in sheet asphalt and bituminous concrete construction relative to the deformation of the surfaces under traffic; and the recovering of bituminous material from aggregates without changing their character.

Surface treatment of gravel roads, crack fillers for concrete pavements, snow removal, road signs, and maintenance accounting form the body of the report of the Committee on Maintenance. This report contains the results of a recent study of bituminous crack fillers together with a complete analysis of the materials used.

The Highway Research Board is not of itself a research agency, but functions as a clearing house for all matters pertaining to highway research. Its functions are threefold: To prepare a comprehensive national program for highway research; to assist existing organizations to coordinate their activities; and to serve as a clearing house for information on completed and current research.

The board has recently enlarged its scope and at the present time is conducting fact-finding surveys on the economic value of reinforcement in concrete roads, the development of earth roads, and an investigation pertaining to culvert pipe.

At a recent meeting of the executive committee of the Highway Research Board, the date of the fifth annual meeting was set as December 3 and 4, 1925. This meeting will be held in the building of the National Academy of Sciences and the National Research Council, B and Twenty-first Streets, Washington, D. C. The program of the meeting will be announced through the technical press at an early date.

The Proceedings of the Fourth Annual Meeting of the board will be sent to highway engineers who make a written request to the Highway Research Board, National Research Council, B and Twenty-first Streets, Washington, D. C.

MOTOR VEHICLE REGISTRATIONS AND REVENUE, ETC.

FOR SIX MONTHS, ENDING JUNE 30, 1925

State	Individually and commercially owned				Official cars and trucks owned by State, etc. [1]	Motor cycles	Registration fees, licenses, permits, etc.		Amount of registration fees paid for—		Grand total motor vehicles first 6 months, 1924	Per cent increase of cars 1925 over 1924	State
	Grand total motor vehicles 6 months, 1925 [1]	Passenger cars [1]	Motor trucks [1]	Taxis, busses, and cars for hire			Total gross receipts	Amount applicable to highway work by or under supervision of State highway department	Passenger cars	Motor trucks			
Alahama	236, 966	194, 487	39, 547	2, 932		361	$1, 882, 557	$1, 845, 525			154, 234	53. 6	Alahama.
Arizona	59, 809	51, 667	8, 142	(2)	858	286	372, 855	372, 855			50, 409	18. 6	Arizona.
Arkansas	148, 981	127, 803	19, 131	2, 047		185	2, 991, 180	2, 641, 180			116, 700	27. 6	Arkansas.
California	1, 317, 825	1, 119, 207	198, 618	(2)	19, 750	9, 787	6, 558, 101	3, 279, 050	$3, 746, 835	$2, 566, 718	1, 184, 015	11. 3	California.
Colorado	213, 891	198, 231	15, 660	(2)		1, 488	1, 311, 368	601, 000	1, 055, 598	189, 750	186, 926	14. 4	Colorado.
Connecticut	216, 746	184, 046	30, 094	2, 606	3, 151	3, 097	5, 069, 455	5, 069, 455	2, 948, 033	1, 080, 491	188, 344	15. 0	Connecticut.
Delaware	35, 600	29, 400	6, 200	(2)		295	613, 474	613, 474	354, 804	126, 484	31, 300	13. 7	Delaware.
Florida	231, 439	192, 000	36, 500	2, 939		730	3, 017, 502	1, 918, 034			164, 112	41. 0	Florida.
Georgia	207, 663	182, 594	24, 968	[3] 101		731	2, 720, 157	2, 650, 051	2, 229, 660	439, 447	177, 030	17. 3	Georgia.
Idaho	[4] 73, 500	[4] 65, 000	[4] 8, 500		970	450	[4] 1, 426, 100	[4] 356, 525			62, 175	18. 2	Idaho.
Illinois	1, 123, 084	981, 436	141, 648	(2)		5, 106	12, 032, 287	12, 007, 383	8, 702, 662	2, 690, 834	986, 480	13. 8	Illinois.
Indiana	523, 783	452, 192	68, 685	[3] 2, 906	308	3, 639	4, 279, 463	4, 139, 170	3, 058, 220	939, 881	581, 899	(7)	Indiana.
Iowa	611, 002	569, 120	41, 882	(2)	2, 000	2, 074	8, 969, 441	8, 162, 191			569, 415	7. 0	Iowa.
Kansas	406, 990	366, 271	40, 719	(2)	1, 884	1, 046	4, 476, 890	4, 297, 814			360, 796	12. 8	Kausas.
Kentucky	233, 828	208, 305	25, 523		1, 147	546	3, 585, 157	3, 541, 157	2, 724, 616	774, 338	203, 028	15. 1	Kentucky.
Louisiana	190, 896	161, 826	29, 070			505	3, 112, 947	3, 112, 947			150, 000	27. 2	Louisiana.
Maine	[5] 126, 200	[5] 104, 750	[5] 18, 295	[5] 3, 155		[5] 1, 200	1, 905, 724	1, 905, 724			110, 282	14. 4	Maine.
Maryland	208, 338	194, 766	10, 600	2, 972		3, 719	2, 065, 033	1, 652, 027	1, 426, 601	250, 447	178, 153	16. 9	Maryland.
Massachusetts	639, 315	551, 563	87, 752	(2)		8, 155	7, 926, 862	[5] 7, 134, 176	5, 186, 065	1, 371, 256	508, 997	25. 6	Massachusetts.
Michigan	798, 460	716, 233	82, 227	(2)	3, 233	2, 540	11, 763, 067	5, 763, 067	8, 407, 037	2, 481, 847	764, 423	4. 4	Michigan.
Minnesota	519, 168	479, 629	39, 189	350		2, 622	9, 037, 663	9, 037, 663	7, 867, 097	913, 553	462, 777	12. 1	Minuesota.
Mississippi	148, 758	133, 882	14, 876			73	1, 413, 894		1, 271, 879	141, 322	112, 099	32. 6	Mississippi.
Missouri	535, 528	486, 146	49, 382	(2)	1, 240	1, 589	3, 250, 206	3, 022, 692			477, 056	12. 2	Missouri.
Montana	83, 950	73, 350	10, 600	(2)	994	1, 922	837, 550		711, 081	125, 134	69, 100	21. 5	Montana.
Nebraska	295, 341	264, 780	30, 561	(2)		868	3, 625, 172	2, 718, 879	2. 951, 971	587, 325	273, 236	8. 1	Nebraska.
Nevada	17, 939	15, 070	2, 869		350	76	188, 891	188, 891			15, 961	12. 4	Nevada.
New Hampshire	73, 120	65, 808	7, 312		286	1, 304	1, 045, 654	990, 051			64, 373	13. 6	New Hampshire.
New Jersey	505, 474	397, 379	98, 387	9, 708		6, 175	9, 198, 365	9, 026, 280	3, 864, 491	2, 836, 669	435, 894	15. 9	New Jersey.
New Mexico	42, 205	40. 550	1, 267	388		184	445, 628	261, 557			34, 951	20. 7	New Mexico.
New York	1, 404, 653	1, 136, 978	237, 233	30, 442	10, 624	15, 737	23, 057, 565	17, 293, 174	14, 057, 217	6, 450, 795	1, 233, 362	13. 8	New York.
North Carolina	[6] 315, 000	[6] 282, 750	[6] 30, 000	[6] 2, 250		[6] 1, 000	[6] 4, 983, 700	[6] 4, 490, 000			[6] 285, 546	10. 3	North Carolina.
North Dakota	126, 106	118, 624	7, 482			321	918, 994	319, 497	822, 731	78, 711	104, 845	20. 3	North Dakota.
Ohio	1, 292, 000	1, 132, 000	160, 000		3, 000	12, 000	11, 969, 324	5, 984, 662			1, 160, 000	11. 3	Ohio.
Oklahoma	420, 000	390, 000	30, 000	(2)		800	4, 112, 723				308, 906	35. 9	Oklahoma.
Oregon	179, 566	166, 107	13, 459	(2)	1, 577	2, 069	4, 844, 310	3, 558, 232	4, 059, 690	683, 825	161, 739	11. 0	Oregon.
Pennsylvania	1, 205, 287	1, 036, 813	162, 307	6, 167	8, 880	13, 180	19, 526, 528	19, 526, 528	10, 755, 719	5, 077, 978	1, 085, 285	11. 1	Pennsylvauia.
Rhode Island	89, 247	72, 603	15, 311	1, 333	587	1, 054	1, 520, 703	1, 446, 713	946, 248	357, 323	78, 413	13. 8	Rhode Island.
South Carolina	141, 208	128, 735	12, 473	(2)	1, 149	108	2, 156, 206	1, 621, 198	1, 653, 991	295, 011	137, 891	2. 4	South Carolina.
South Dakota	150, 335	138, 257	11, 680	398	712	280	2, 279, 479	1, 006, 904	2, 039, 291	234, 768	126, 813	18. 5	South Dakota.
Tennessee	218, 735	196, 389	20, 409	1, 937		554	2, 808, 119	2, 808, 119			182, 723	19. 7	Tennessee.
Texas	848, 661	773, 464	72, 505	2, 692		2, 201	12, 081, 524	8, 676, 292			661, 919	28. 2	Texas.
Utah	76, 410	66, 850	9, 560			557	496, 851	437, 200			67, 133	13. 8	Utah.
Vermont	60, 424	56, 083	4, 341			576	1, 350, 639	1, 279, 362	1, 045, 223	110, 520	52, 614	14. 8	Vermont.
Virginia	225, 800	222, 500	3, 300			1, 500	3, 813, 317	3, 617, 650	3, 100, 951	484, 711	225, 343	. 2	Virginia.
Washington	293, 589	250, 257	40, 331	3, 001	3, 789	2, 196	4, 615, 950	4, 545, 326	3, 392, 992	978, 430	265, 541	10. 6	Washington.
West Virginia	184, 200	158, 400	22, 450	3, 350	1, 135		2, 898, 418	2, 898, 418	2, 030, 637	493, 638	154, 772	19. 0	West Virginia.
Wisconsin	534, 662	476, 188	58, 474	(2)	437	2, 864	7, 800, 438	7, 420, 000			473, 913	12. 8	Wisconsin.
Wyoming	42, 600	38, 000	4, 600		200	200	437, 000	437, 000			38, 200	11. 4	Wyoming.
District of Columhia	82, 427	71, 158	10, 631	638	1, 939	1, 324	105, 278	105, 278			72, 954	13. 0	District of Columbia.
Total	17, 716, 709	15, 519, 647	2, 114. 750	82, 312	70, 200	119, 274	226, 899, 709	183, 780, 371	[8] 100, 111, 340	[8] 32, 761, 206	15, 552, 077	13. 9	Total.

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[1] Net number of cars and trucks registered shown when possible, excluding reregistrations, nonresident, and official exempt cars and trucks.

[2] Recorded with private cars or trucks.

[3] Motor busses only.

[4] Estimated from 3 months' return.

[5] Approximate.

[6] Estimate for registration year ending June 30.

[7] Decrease.

[8] Only data for 27 States shown

THE GASOLINE TAX IN THE FIRST SIX MONTHS OF 1925

State	Date of rate change 1925	Tax rate per gallon		Gross receipts Jan. 1 to June 30, 1925	Distribution of gross receipts			State
					Applied to road work			
		Jan. 1, 1925	June 30, 1925		Supervision State highway department	County and local road funds	Miscellaneous purposes	
		Cents	*Cents*					
Alabama		2	2	$963,520		$958,604	Collection eosts, $4,916	Alabama.
Arizona		3	3	401,438	$200,719	200,719		Arizona.
Arkansas		4	4	1,520,789	1,170,789	350,000		Arkansas.
California		2	2	7,514,667	3,753,584	3,753,583	Collection costs, about $7,500	California.
Colorado		2	2	914,962	434,607	434,607	5 per cent administration, $45,748	Colorado.
Connecticut		1	1	527,671	527,671	0		Connecticut.
Delaware		2	2	157,291	157,291	0		Delaware.
Florida		3	3	2,731,387	1,431,843	715,921	Refunds and expenses, $583,623	Florida.
Georgia		3	3	1,837,203	612,401	612,401	State general fund, $612,401	Georgia.
Idaho		2	2	339,466	339,466	0		Idaho.
Illinois	No tax.	0	0					Illinois.
Indiana	4/1	2	3	[1] 3,214,588	2,246,438	962,876	Collection costs, $5,274	Indiana.
Iowa	4/16	0	2	978,855	326,285	652,570		Iowa.
Kansas	5/1	0	2	892,208	713,766	178,442		Kansas.
Kentucky		3	3	1,347,499	1,347,499	0		Kentucky.
Louisiana		2	2	1,091,691	1,091,691	0		Louisiana.
Maine		1	[2] 1	202,477	202,477	0		Maine.
Maryland		2	2	870,975	[3] 852,628	0	Refunds and collection costs, $18,347	Maryland.
Massachusetts	No tax.	0	0					Massachusetts.
Michigan	2/1	0	2	3,385,500	3,385,500	0		Michigan.
Minnesota	5/1	0	2	1,260,839	1,260,839	0		Minnesota.
Mississippi		3	3	1,079,701	523,495	523,495	Collection costs, $32,711	Mississippi.
Missouri	1/1	0	2	2,169,902	1,991,902	0	Collection costs and refunds, $178,000	Missouri.
Montana		2	2	226,229	36,229	117,542	State general fund, $72,458	Montana.
Nebraska	4/1	0	2	681,215	673,215	0	Collection costs, $8,000	Nebraska.
Nevada		2	[4] 2	100,228	50,114	50,114		Nevada.
New Hampshire		2	2	263,027	263,027	0		New Hampshire.
New Jersey	No tax.	0	0					New Jersey.
New Mexico	3/17	1	3	185,019	175,768	0	Collection costs, $9,251	New Mexico.
New York	No tax.	0	0					New York.
North Carolina	3/5	3	4	2,782,242	2,767,467	0	Collection costs, $14,775	North Carolina.
North Dakota		1	1	250,150	50,150	0	State general fund, $200,000	North Dakota.
Ohio	4/21	0	2	2,515,435	1,131,946	754,631	Municipal roads, $628,859	Ohio.
Oklahoma	3/23	2½	3	2,150,549	1,351,929	798,620		Oklahoma.
Oregon		3	3	1,336,593	1,275,664	0	Expense and refunds, $60,929	Oregon.
Pennsylvania		2	2	4,657,752	375,440	1,178,852	State general fund, $3,103,459	Pennsylvania.
Rhode Island	4/29	0	1	45,848	45,848	0		Rhode Island.
South Carolina	3/23	3	5	1,583,910	817,612	600,380	State general fund, $165,918 [5]	South Carolina.
South Dakota	3/10	2	3	772,937	772,937	0		South Dakota.
Tennessee	2/9	2	3	1,380,898	690,449	690,449		Tennessee.
Texas		1	1	2,113,572	1,585,179	0	Free school fund, $528,393	Texas.
Utah	4/1	2½	3½	429,363	428,000	0	Collection costs, about $1,363	Utah.
Vermont	2/26	1	2	171,867	171,867	0		Vermont.
Virginia		3	3	1,681,786	1,121,191	560,595		Virginia.
Washington		2	2	1,445,786	1,437,847	0	Collection costs, $7,939	Washington.
West Virginia		2	[6] 2	632,280	632,280	0		West Virginia.
Wisconsin	4/1	0	2	779,838	779,838	0		Wisconsin.
Wyoming	4/1	1	2½	130,190	129,560	0	Collection costs, $630	Wyoming.
District of Columbia		2	2	389,391	385,391	0	Refunds, $4,000	District of Columbia.
Total				60,108,734	39,719,839	14,094,401		Total.

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[1] Refunds excluded.
[2] Changed to 3 cents on July 11, 1925.
[3] Baltimore received one-fifth of this amount, $170,525.
[4] Changed to 4 cents on July 1, 1925.
Part of disposal of first three months: New law excludes State general fund.
[6] Changed to 3½ cents on July 1, 1925.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

AUGUST 31, 1925

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863,197.88	611.8	$ 573,244.10	$ 283,034.64	26.9	$ 14,838,392.66	$ 7,193,504.05	777.5	$ 300,107.34	$ 146,749.34	24.8	$ 2,322,170.11	Alabama
Arizona	9,580,133.43	5,016,119.94	613.8	237,926.80	145,397.06	7.1	2,043,979.33	1,270,492.41	157.2	276,880.71	180,420.97	35.6	1,948,910.62	Arizona
Arkansas	13,310,190.08	5,380,181.73	1,048.9	127,585.44	61,163.46	8.3	7,415,243.72	3,357,050.45	390.6	1,622,859.12	778,529.39	131.8	750,971.97	Arkansas
California	22,346,176.98	10,719,249.81	894.8	1,026,130.48	509,752.08	24.4	10,182,004.31	4,983,478.43	261.1	1,886,658.50	761,081.95	89.3	2,614,546.83	California
Colorado	11,876,703.94	6,067,844.34	651.2	352,483.31	182,668.63	16.5	4,231,281.75	2,115,125.88	203.6	596,482.73	364,874.34	34.3	2,224,944.81	Colorado
Connecticut	4,558,839.29	1,819,368.66	101.6	575,452.78	171,947.14	9.5	1,909,405.24	609,235.48	28.8				1,259,701.72	Connecticut
Delaware	4,281,559.81	1,496,190.65	107.1	383,163.86	180,583.85	12.1	76,658.90	31,961.10	0.2	980,042.66	388,784.75	22.7	10,912.65	Delaware
Florida	2,959,273.72	1,405,487.97	96.3				8,787,223.95	4,279,162.51	251.8	776,683.27	295,728.85	11.8	1,207,389.66	Florida
Georgia	20,166,002.37	9,408,366.46	1,478.3	862,445.36	423,355.93	50.9	10,828,976.87	5,350,287.49	706.6	1,181,844.78	565,915.77	81.0	704,838.35	Georgia
Idaho	9,394,676.80	4,815,332.26	500.1	52,764.61	29,828.56	14.9	2,537,676.75	1,600,882.02	142.5	766,334.43	470,008.62	69.2	706,986.54	Idaho
Illinois	40,010,481.10	18,640,076.28	1,236.2	539,655.66	267,490.93	19.8	8,449,891.73	4,091,834.64	284.6				3,657,180.15	Illinois
Indiana	13,639,172.65	6,562,455.68	422.1	1,071,523.96	516,206.70	33.6	15,564,525.88	7,306,154.67	439.0	848,584.16	418,630.58	23.2	1,465,017.37	Indiana
Iowa	27,272,285.21	11,107,492.99	1,996.9	102,280.50	39,495.95	16.3	8,049,447.05	3,575,941.22	461.5	731,202.22	364,295.00	109.9	2,337,868.84	Iowa
Kansas	26,389,895.77	9,755,273.32	831.4	1,920,935.60	783,687.96	75.0	11,491,869.89	4,952,358.43	563.9	797,516.87	325,967.24	42.9	1,573,958.05	Kansas
Kentucky	14,832,324.28	6,205,994.59	584.9	553,446.24	219,076.66	17.8	7,893,919.22	3,522,706.87	281.0	590,422.33	295,211.15	40.0	1,553,010.73	Kentucky
Louisiana	11,939,424.97	5,279,870.86	927.8	875,723.58	411,312.04	64.9	2,772,603.62	1,325,240.49	108.8	356,130.73	178,055.36	2.8	1,077,155.25	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	173,275.15	85,733.59	6.8	1,311,335.31	561,480.38	42.2				1,235,169.70	Maine
Maryland	8,132,505.90	3,849,383.15	294.4				2,070,450.66	884,007.47	55.7	1,139,721.78	646,339.26	75.3	10,703.12	Maryland
Massachusetts	14,047,656.22	5,467,661.28	300.6	303,451.64	89,080.00	4.4	5,416,418.63	1,414,104.74	83.3	892,826.32	318,076.37	12.2	1,730,748.61	Massachusetts
Michigan	16,234,000.80	7,328,316.91	612.6	1,957,578.61	929,997.15	88.8	13,133,359.02	6,127,491.80	394.5	1,004,764.44	389,378.76	0.6	3,349,762.38	Michigan
Minnesota	30,415,685.88	12,738,642.04	2,721.2				9,146,490.80	4,021,900.00	781.0	1,021,808.58	208,800.00	72.0	492,269.96	Minnesota
Mississippi	10,292,285.79	4,988,702.73	803.4	952,131.44	475,374.67	67.5	8,104,412.27	4,047,450.48	439.0	1,074,297.43	537,118.68	68.6	786,168.44	Mississippi
Missouri	17,358,156.57	8,219,411.43	1,118.9	1,448,054.02	895,155.96	55.7	26,255,129.47	10,815,100.30	819.1	1,425,163.43	160,507.15	37.5	489,404.17	Missouri
Montana	10,156,600.41	5,317,523.15	921.6	448,444.40	390,460.39	49.5	1,703,354.88	1,187,274.00	134.1	971,521.92	542,524.18	116.6	4,435,443.28	Montana
Nebraska	9,306,374.36	4,389,523.50	1,570.6	795,963.75	396,021.48	66.8	7,638,135.16	3,800,068.85	782.5	2,943,547.56	1,466,650.78	297.4	3,004,832.39	Nebraska
Nevada	4,917,465.69	3,088,299.78	357.3	176,699.88	131,175.94	18.2	5,120,750.89	4,289,231.40	431.6	125,098.64	109,736.51	22.5	228,463.37	Nevada
New Hampshire	4,165,687.86	1,986,225.87	208.1				1,191,106.61	561,352.43	37.9	50,919.49	22,558.87	2.1	233,718.83	New Hampshire
New Jersey	11,961,357.45	3,820,679.99	219.1	568,121.43	131,911.08	8.1	9,554,745.73	3,324,061.87	67.8	658,080.63	205,446.25	15.7	50,612.83	New Jersey
New Mexico	8,717,999.18	4,914,070.61	1,081.3	1,473,170.37	987,112.81	134.7	2,847,896.13	1,835,025.39	259.9	405,214.35	253,070.82	29.6	1,795,842.37	New Mexico
New York	28,597,769.67	12,229,076.53	831.5	2,702,139.52	1,051,190.12	66.8	29,386,225.75	9,698,510.10	596.0	10,127,400.00	2,679,401.45	177.8	4,739,850.80	New York
North Carolina	21,014,450.41	8,746,454.59	1,119.8	1,114,845.28	437,085.30	34.4	7,793,610.79	3,363,112.05	191.5	2,055,720.25	884,513.17	58.5	577,486.89	North Carolina
North Dakota	10,829,263.82	5,268,930.47	1,917.5	310,319.25	155,029.12	92.5	3,616,874.00	1,824,088.87	465.4	946,269.64	473,134.67	113.8	1,833,755.87	North Dakota
Ohio	41,572,252.81	15,244,993.93	1,191.1	1,187,261.09	392,270.66	34.1	10,288,633.77	3,778,391.31	308.6	4,164,856.38	1,554,718.51	113.4	1,974,384.59	Ohio
Oklahoma	20,787,024.94	9,672,890.34	852.2	1,529,982.01	684,504.73	52.3	5,501,764.53	2,626,747.20	259.3	1,766,875.55	849,421.20	85.2	473,978.53	Oklahoma
Oregon	14,388,188.70	7,142,364.63	794.6	215,169.50	124,212.68	10.2	2,829,653.13	1,630,589.68	127.2	1,120,927.02	505,715.55	82.1	293,519.46	Oregon
Pennsylvania	43,054,835.19	15,222,023.97	850.3	651,585.18	191,522.50	11.8	32,501,905.46	8,640,032.64	564.2	6,849,582.03	2,001,018.33	152.1	937,263.56	Pennsylvania
Rhode Island	2,528,496.20	1,119,688.09	64.8				2,450,499.34	654,039.59	35.3	82,759.50	27,900.00	1.9	600,315.32	Rhode Island
South Carolina	11,163,347.84	5,121,267.54	1,235.9	959,030.47	315,193.36	49.2	5,649,786.65	2,658,789.19	325.0	1,002,147.52	412,092.84	41.0	232,188.07	South Carolina
South Dakota	12,091,434.67	5,989,879.00	1,447.9	861,885.50	412,489.09	92.8	7,137,671.31	3,405,497.96	1,007.9	184,120.15	92,060.06	27.5	44,665.89	South Dakota
Tennessee	13,789,140.98	6,732,079.77	497.9	2,163,355.52	1,055,751.63	70.4	9,930,886.09	4,436,772.39	352.4	128,336.55	64,168.27	10.8	1,363,400.04	Tennessee
Texas	54,120,970.83	21,057,940.12	3,907.1	1,805,447.98	761,222.85	153.8	23,591,944.63	9,921,184.80	1,362.1	3,904,117.08	1,520,469.66	296.8	2,818,695.87	Texas
Utah	8,259,159.41	3,818,836.91	423.1	132,178.99	99,880.35	5.9	2,876,066.20	1,937,063.10	220.0	350,634.01	257,973.13	35.4	866,784.51	Utah
Vermont	3,015,174.51	1,452,894.45	107.8	22,599.35	11,299.67	0.2	1,195,915.15	572,446.74	26.4	586,102.70	205,752.15	12.9	660,488.99	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	1,720,105.73	850,257.20	80.8	8,491.568.90	3,725,275.21	271.9	2,161,254.07	979,352.48	76.1	228,768.91	Virginia
Washington	13,352,504.18	6,117,211.87	526.7	1,049,945.31	437,338.64	25.0	2,094,224.66	988,900.00	102.3	1,130,579.45	509,000.00	35.3	963,245.49	Washington
West Virginia	7,343,200.86	3,230,293.33	326.7	252,664.30	123,013.89	12.8	4,944,376.90	2,053,284.68	135.0	3,183,826.54	1,144,830.33	89.3	7,152.77	West Virginia
Wisconsin	21,807,140.91	8,919,640.62	1,451.7				4,734,981.91	2,328,946.59	217.6	1,236,349.35	607,256.50	82.8	3,712,709.29	Wisconsin
Wyoming	8,809,819.33	4,739,096.67	982.0	473,184.83	278,096.58	42.1	3,758,189.67	2,366,555.53	256.9	99,681.78	63,995.00	12.1	182,936.22	Wyoming
Hawaii							342,277.22	97,440.00	6.5				637,088.00	Hawaii
TOTALS	740,140,790.82	325,654,346.00	41,898.3	34,714,362.86	15,927,392.60	1,733.2	379,693,773.29	165,141,632.88	16,229.8	64,406,373.89	25,218,254.25	2,974.2	66,308,374.27	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 101,031,316.39 Federal aid $ 44,794,856.73 Miles 4,390.6

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 6, NO. 8 ▽ OCTOBER, 1925

General View of Deflectometer Used in Tests of Six-Wheel Trucks

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1925

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

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THE SIX-WHEEL TRUCK AND THE PAVEMENT

TESTS CONDUCTED BY THE UNITED STATES BUREAU OF PUBLIC ROADS TO DETERMINE THE RELATIVE EFFECT OF FOUR AND SIX WHEEL TRUCKS

Reported by **L. W. TELLER**, Associate Engineer of Tests, United States Bureau of Public Roads

TESTS conducted recently by the United States Bureau of Public Roads indicate definitely that the tensile stress set up in a concrete pavement by a six-wheel truck is only about half as great as the stress produced by a four-wheel truck of the same gross load. The tests show also that the stress produced in the pavement by the six-wheel vehicles is a function of the load on the wheels and not of the axle spacing. This seems to be true for all spacings of the rear axles greater than 3 feet, there being some indication that when the wheels are closer together than 36 inches the stress produced in the pavement may be increased.

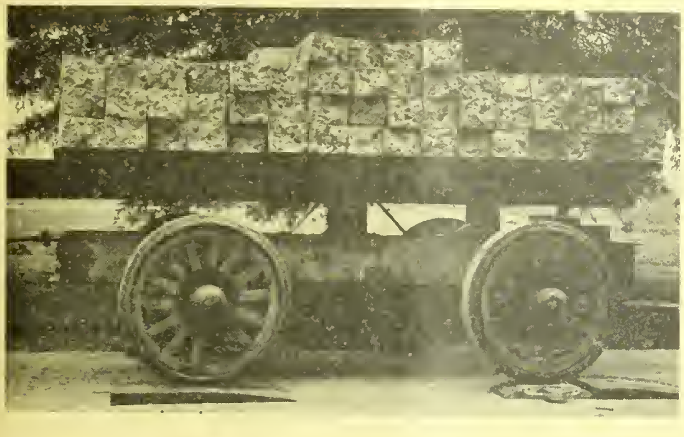

FIG. 1.—Special trailer with 86-inch axle spacing. The rear wheels are resting on the loadometers, the front wheels on the leveling platforms

It is also shown that under six-wheel as well as four-wheel trucks the maximum tensile stress occurs in the bottom of the slab, regardless of the axle spacing. Even though there is counterflexure of the pavement between the wheels, the tension developed in the top of the slab is of less magnitude than the tension developed in the bottom of the slab directly under the wheels. This latter tension, therefore, is the critical stress for six-wheel as well as four-wheel vehicles.

Loads passing over the pavement 21 inches from the edge were found to produce an average stress less than 50 per cent as great as that produced by the same loads passing 9 inches from the edge.

The pavement on which the tests were made was a concrete slab of uniform thickness, and the maximum deformation of this slab was found to occur along the edge under both four and six wheel vehicles. Subjected to loads of varying magnitude applied at the same point, both the deflection and the deformation of the slab were found to be proportional to the load.

The six-wheel truck is the motor-vehicle designers' solution of the problem presented by the demand for vehicles which will carry heavy "pay loads" without violation of the restrictive regulations adopted for the protection of the highways. The regulations are the outgrowth of experience and research, which show that heavy concentrated loads unwarrantably increase the cost of pavement construction and maintenance. Such regulation limits the gross load, the axle load, the wheel load, the load per inch of tire width supported by the pavement, or several of these factors.

Facing such restrictions, the motor-vehicle designers have sought to increase the legal carrying capacity of the single vehicle by using four wheels instead of two at the rear, where the bulk of the load is carried, thus doubling the number of axles, wheels, and inches of tire width by which the load is transmitted to the pavement. Such vehicles comply with the regulations and can legally carry heavier loads than the four-wheel vehicles, and the increase in "pay load" accommodated is much greater than the increase in dead load necessitated by the new design.

While it has seemed reasonable to assume that the addition of the extra pair of wheels would accomplish the purpose intended, the Bureau of Public Roads has felt that the effect upon the pavement should be determined experimentally, and to that end the tests reported in this article have been conducted to study the behavior of a concrete pavement under both six and four wheel trucks.

OBJECTS AND METHODS OF TEST

The principal objects of the tests were to determine:

1. The relative stresses produced in the pavement by four and six wheel vehicles having (*a*) the same gross load, and (*b*) the same wheel load.
2. The effect of the axle spacing of six-wheel vehicles on the stress produced in the pavement.

In order to study the effect of various axle spacings a special trailer was constructed consisting of a frame provided with two axles, which could be spaced at any distance from 3 to 10 feet apart, the load being equally supported by all four wheels. The front end of the platform that carried the cargo rested on a knife-edge placed transversely on the rear end of the motor truck which was used to move the trailer. Thus the essentials of a six-wheel vehicle were reproduced and axle spacings of from 3 to 10 feet could be had at will. As the trailer was to be run only at very low speeds over a smooth pavement, no springs were provided. The wheels were equipped with 40 by 6 inch single solid rubber truck tires. Figure 1 shows the general appearance of the unit. The four-wheel, 5-ton capacity truck (fig. 2), equipped with similar tires, was used for comparative tests. In addition to the equipment described above two other trucks were used, one a six-wheel and one a four-wheel, class B Army truck of

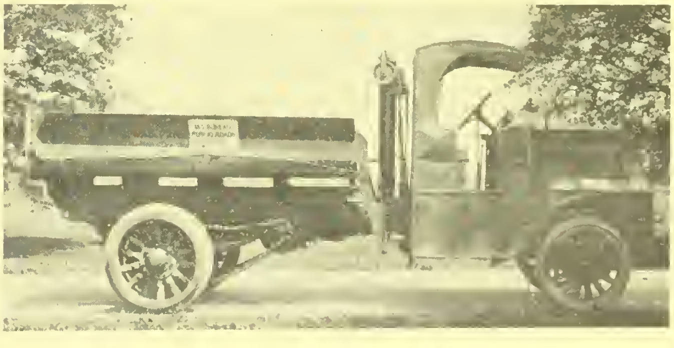

FIG. 2.—Four-wheel truck with solid tires

identical design except for the rear end construction. These two trucks, which were equipped with pneumatic tires, are shown in Figures 3 and 4.

Two "loadometers" were used to measure the wheel loads, these devices having been calibrated prior to their use in this investigation. The vehicle being weighed was supported in a level position by small wheel platforms, as shown in Figure 1.

FIG. 3.—Six-wheel truck with pneumatic tires

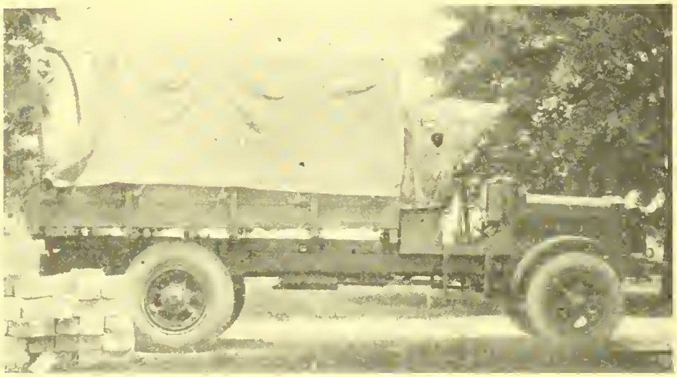

FIG. 4.—Four-wheel truck with pneumatic tires

The test pavement was a 6-inch plain concrete slab of uniform cross section. The width of the slab was 18 feet and the length between transverse joints was 30 feet. The tests were made at one corner of this slab, the transverse joint being a poured joint, without expansion material.

The deflection data were secured with an instrument called a "deflectometer," which, as shown on the cover, consists of a series of pen arms supported by the pavement. Each pen arm traces a continuous record on a moving paper of the deflection of the slab at that point as the truck wheel moves over the slab. Thus, each curve is an influence line of deflection for the particular point at which the pen arm is supported. The vertical movement of the slab is multiplied fifty times on these curves by a direct lever action. From a series of such curves it is possible to obtain the elastic curve of the pavement for any position of the wheel. The apparatus is provided with electric contacts which record on the same paper the instant at which the truck passes a given point. The speed of the truck and of the paper are determined by a time-recording mechanism which is simply a pendulum whose known period is recorded electrically on the moving paper record. Details of this part of the apparatus are shown in Figure 6.

Graphic strain gauges, illustrated in Figure 6,[1] were installed along the upper and lower edges of the slab in the positions shown in Figure 7. These gauges, which recorded the maximum compressive and tensile deformations occurring in the outer fibers of the concrete pavement during each passage of the truck, were protected by a canvas sunshade during the tests.

Two traffic lines were painted on the pavement—one 9 inches, the other 21 inches from the edge. During the test runs the outer wheels of the vehicle were kept centered over one or the other of these lines. From three to five runs were made under each test condition, depending upon the manner in which the recording apparatus functioned.

Complete data were obtained on nearly 100 runs, the conditions being varied according to the following program:

I. Four-wheel trucks:
  (*a*) Pneumatic tires, 44 by 10 inches, 9 inches from edge of pavement.
    1. 4,000-pound wheel load.
    2. 8,000-pound wheel load.
  (*b*) Single solid tires, 40 by 6 inches, 9 and 21 inches from edge of pavement, with 4,000, 5,000, 6,000, 7,000, and 8,000 pound wheel loads.

II. Six-wheel trucks:
  (*a*) Pneumatic tires, 38 by 9 inches, with 50-inch axle spacing, 9 inches from edge of pavement.
    1. 4,000-pound wheel load.
    2. 4,300-pound wheel load.
  (*b*) Pneumatic tires, 40 by 6 inches, with 4,350-pound wheel load.
    1. Nine inches from edge of pavement and 36, 40, 44, 48, 54, 60, 72, 86, and 118 inch axle spacings.
    2. Twenty-one inches from edge of pavement and 36, 40, 44, and 54 inch axle spacings.

ELASTIC CURVES DEVELOPED FROM DEFLECTION MEASUREMENTS

As previously remarked, the deflection data obtained consisted of deflection influence lines for a series of points along the edge of the pavement. Each of these curves showed the deflection of a point of the pavement as the truck passed over it. By relating the movement of the load to the curve, the deflection of that particular part of the pavement for any position of the load can be obtained. With a series of such influence curves representing the deflection of a series of points along the edge of the pavement, it is possible to determine the form assumed by the pavement under the load, or, in other words, the elastic curve of the pavement.

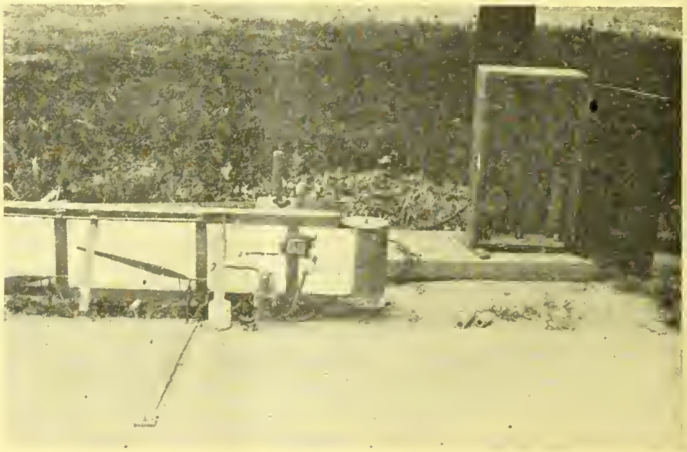

FIG. 5.—Details of pendulum and road contacts of deflectometer

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[1] For a description of the graphic strain gauge see Engineering News-Record, vol. 90, No. 13, Mar. 29, 1923, p. 575.

FIG. 6.—Graphic strain gauge developed by the Bureau of Public Roads. The gauge is inserted in a slot cut into the concrete with the two points bearing against the concrete at the ends of the slot. Deformation of the concrete causes a throw of the lever, which causes a point at its end to trace a line on a smoked-glass slide. The length of the line is proportional to the deformation

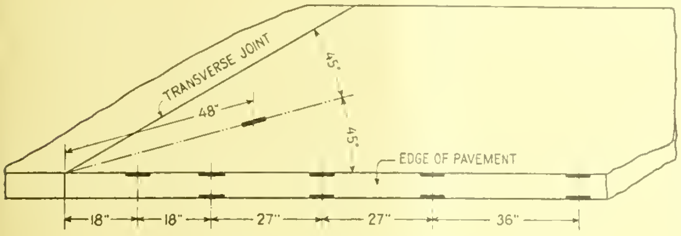

FIG. 7.—Layout of graphic strain gauges

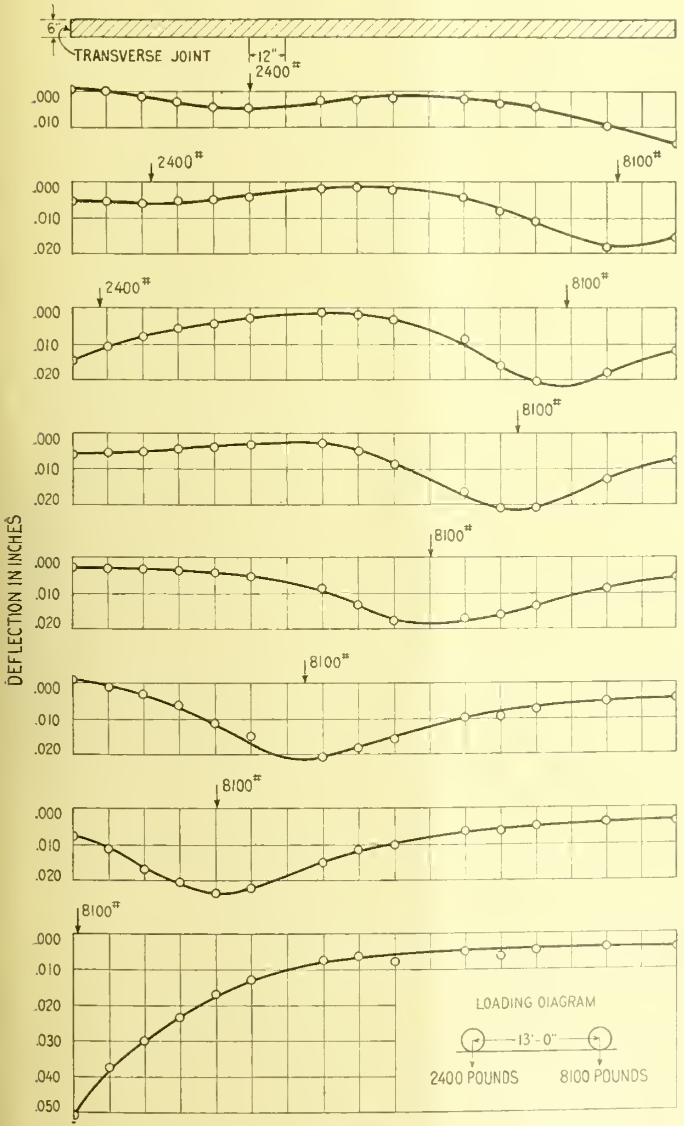

FIG. 8.—Elastic curves of 6-inch concrete pavement under 4-wheel truck in various positions

While so far as this investigation is concerned the elastic curve of the pavement is not of primary importance, two such curves have been included (figs. 8 and 9) to show the extent and nature of the deflection which actually occurs under the four and six wheel trucks respectively.

From these curves it will be seen that the deflection is directly proportional to the load. For instance, a 2,400-pound wheel load at the corner of the slab produced a maximum deflection of 0.015 inch. The rear wheel load of 8,100 pounds passing the same point produced a maximum deflection of 0.051 inch. The same is true for deflections along the edge of the slab away from the corner, although the deflection is only about 40 per cent as great. Very little trouble was experienced with the deflection-measuring apparatus and the quality of the records obtained was excellent.

The data from the strain gauges consisted of graphic records showing the maximum tensile and compressive deformations occurring at the gauge position for each load application. These records in each case were reduced to the maximum unit deformation, an average value over the 6-inch gauge length, and were corrected slightly for the position of the gauge with respect to the surface of the pavement by locating the position of the neutral plane of the pavement and assuming a straight-line variation of stress. These corrections were usually small, as the gauges were placed as near the surface of the slab as possible.

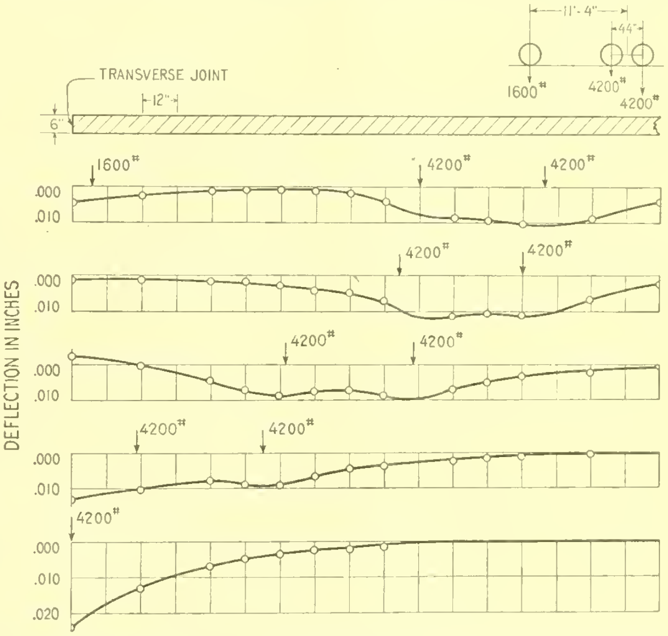

FIG. 9.—Elastic curves of 6-inch concrete pavement under 6-wheel truck in various positions

Examining the data from all of the runs with the various vehicles, the stress produced along the edge of the pavement by a wheel load 21 inches from the edge is found to be 47 per cent of that produced by the same load 9 inches from the edge. This percentage varied from 43 to 51 per cent for the various groups of comparable results.

COUNTERFLEXURE UNDER SIX-WHEELERS FAILS TO PRODUCE HIGH TOP STRESSES

The variation in stress along the edge of the pavement for a given load is shown by Figure 10. The diagram shows the maximum edge stresses at the several points resulting from the load as it passes along the edge.

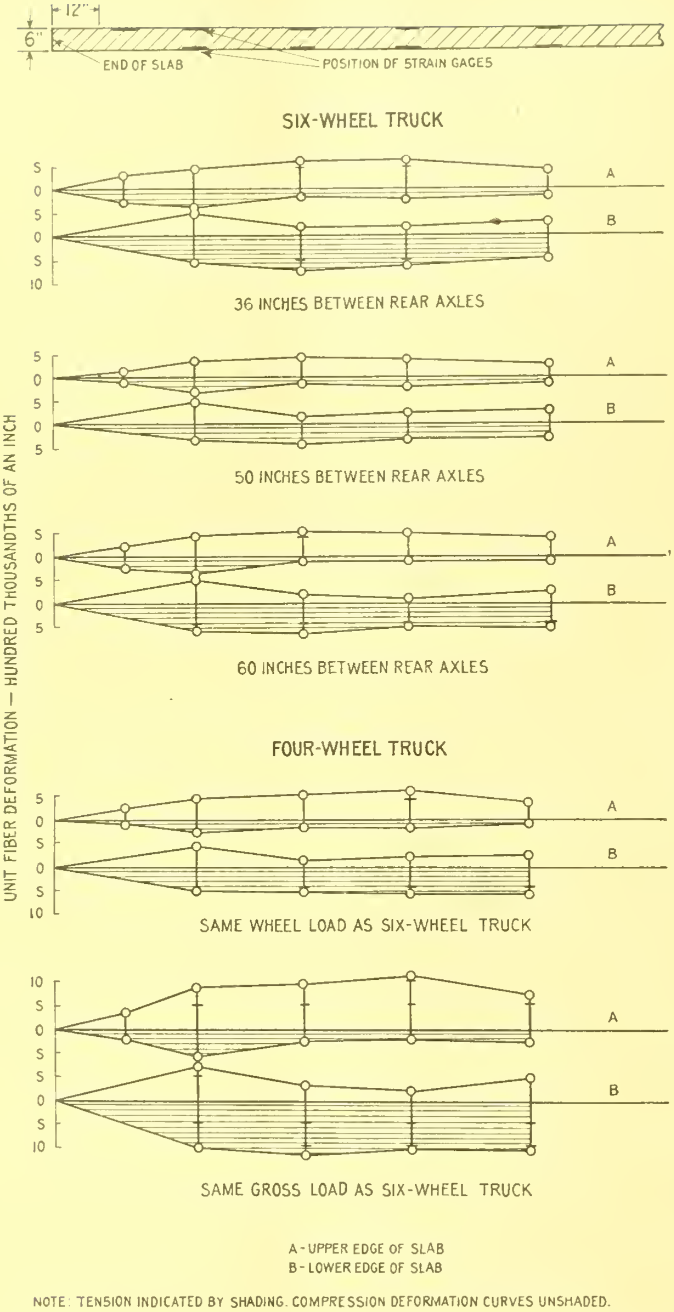

Fig. 10.—The variation in stress along the edge of the pavement resulting from the passage of 6-wheel and 4-wheel trucks. The stresses indicated are the maxima produced at the several points by the loads in any position

The variation has the same characteristics that were shown in a number of tests previously conducted by the Bureau of Public Roads. The significance of these diagrams lies in the fact that they show that for six-wheel trucks the maximum tensile stress occurs in the bottom of the slab regardless of the axle spacing. This means that even though there is counterflexure of the pavement between the wheels the tension developed in the top of the slab is of less magnitude than the tension developed in the bottom of the slab directly under the wheels. This latter tension, then, is the critical stress for the six-wheel as well as the four-wheel vehicle.

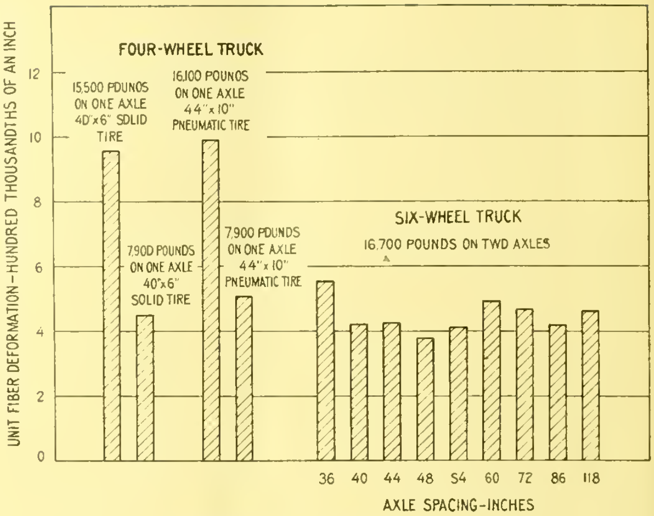

Fig. 11.—Unit fiber deformations of 6-inch concrete slab by 4-wheel and 6-wheel trucks

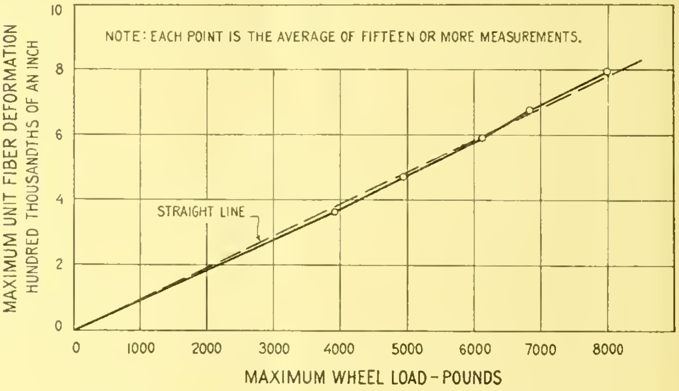

Fig. 12.—The relation between wheel load and fiber deformation

This leads to a study of the relative fiber deformations produced by four-wheel and six-wheel vehicles, having, first, the same gross loads, and, second, the same wheel loads. These data are presented graphically in Figure 11. The difference between the two four-wheel trucks was only in the area of contact between the tire and the pavement due to the type of tire, and it is obvious that this has no appreciable effect on the stress. The data indicate that for a given gross load the six-wheel truck will cause only about one-half the tensile deformation produced by the four-wheel truck. This seems to be true for all axle spacings with the possible exception of the 36-inch, for which the recorded deformation is slightly greater. This may be due to some peculiarity of this particular test, although it seems more probable that with wheels but 36 inches apart the condition is beginning to approach that of a concentrated load. However, as it is doubtful if such a close axle spacing will be met in practice, this small increased stress would not appear to be important. The significant indication is that the stress in the pavement produced by a six-wheel truck is a function of the wheel load and not of the axle spacing.

Sufficient data were obtained in these tests to show the relation between wheel load and fiber deformation. These data are shown graphically in Figure 12. Fifteen or more measurements were averaged for each of five-wheel loads (4,000, 5,000 6,000, 7,000, and 8,000 pounds). These average values lie along a straight line, with a maximum deviation of less than 4 per cent.

(Continued on page 184)

RAILROAD ABANDONMENTS AND THEIR RELATION TO HIGHWAY TRANSPORTATION

Reported by HENRY R. TRUMBOWER, Economist, UnitedStates Bureau of Public Roads

IN 1916 there were 254,037 miles of railroad in the United States. That was the peak year. Every year since has shown a net decrease in the mileage, so that at the end of 1923 there were only 250,222 miles, or 3,815 miles less than the peak mileage. As during the seven-year period there was some mileage added as well as abandoned, the actual mileage abandoned was greater than 3,815 miles.

This reduction in the railroad mileage of the country has been called to the attention of the public by some writers as an indication of a condition of general stagnation of the railroad business. More recently it has been intimated to be the result of the increased operation of motor vehicles over the improved highways.

In order to determine the significance and economic importance of these abandonments, and especially to ascertain the influence of motor vehicle operation, the Bureau of Public Roads has made a study of the circumstances surrounding the abandonments since 1920, the results of which lead to the conclusion that in but few instances has highway competition been the primary cause of the abandonment. It would be difficult, if not impossible, to determine the causes of the curtailments made before 1920, but since that year the information has been made available as a result of the passage of the transportation act.[1] That act provides for the issuance of certificates of public convenience and necessity by the Interstate Commerce Commission in all cases of contemplated abandonment of lines by any railroad. The facts involved in these abandonment proceedings are fully set forth in the published opinions of the commission, and in that way are made matters of public information.

AVERAGE ABANDONMENT 500 MILES A YEAR SINCE 1920

From 1920, when the act was passed, to May 1, 1925, the Interstate Commerce Commission granted certificates of public convenience and necessity authorizing the abandonment of 120 lines and branches aggregating 2,438.95 miles. The details follow in Table 1.[2]

TABLE 1.—*Mileage of railroad lines and branches abandoned from 1920 to 1925*

Year	Lines		Branches		Total	
	Number	Miles	Number	Miles	Number	Miles
From Nov. 1, 1920	1	30.30	3	23.97	4	54.27
1921	14	548.12	20	144.82	34	692.94
1922	9	387.56	15	196.05	24	583.61
1923	6	305.30	14	152.39	20	457.69
1924	7	180.70	19	248.38	26	429.08
To May 1, 1925	4	157.65	8	63.71	12	221.36
Total	41	1,609.63	79	829.32	120	2,438.95

The line abandonments consist of all those which involved the abandonment of a whole line of railroad; the branch abandonments include those which involved the abandonment of a portion of a company's line of railroad or one of its branches. These authorizations by the commission covered during this period 41 lines of railroad totaling 1,609.63 miles and 79 branches constituting 829.32 miles. Of all the abandonments, in other words, 66 per cent were branches and 34 per cent were lines. The branches, however, constituted only 34 per cent of the abandoned mileage and the lines constituted 66 per cent. The average length of an abandoned line was 39.25 miles and of an abandoned branch only 10.49 miles.

EAST NORTH CENTRAL STATES SHOW LARGEST ABANDONMENT

The abandoned mileage is not confined to any particular section of the country; in every State except eight there were some abandonments. The States in which no mileage was relinquished were Delaware, Idaho, Kansas, Kentucky, Maryland, Missouri, Nebraska, and Utah. The distribution by geographic sections is shown in Table 2.

TABLE 2.—*Distribution of railroad abandonments by geographic sections* [1]

Group of States	Lines			Branches			Total		
	Number	Miles	Average length	Number	Miles	Average length	Number	Miles	Average length
New England				11	54.47	4.95	11	54.47	4.95
Middle Atlantic	1	37.00	37.00	7	54.65	7.81	8	91.65	11.46
East North Central	8	633.78	79.22	18	176.48	9.81	26	810.26	31.16
West North Central	2	153.05	76.52	7	64.08	9.15	9	217.13	24.13
South Atlantic	10	220.95	22.09	8	27.65	3.45	18	248.60	13.81
East South Central	4	166.61	41.65	4	87.98	21.99	8	254.59	31.82
West South Central	10	233.34	23.33	10	120.56	12.06	20	353.90	17.69
Mountain	2	33.50	16.75	8½	[2] 118.37	13.92	10½	[2] 151.87	14.46
Pacific	4	131.40	32.84	5½	[2] 125.08	22.74	9½	[2] 256.48	26.99
Total	41	1,609.63	39.25	79	829.32	10.49	120	2,438.95	20.32

[1] New England States: Maine, New Hampshire, Vermont, Massachusetts, Connecticut, and Rhode Island. Middle Atlantic States: New York, New Jersey, and Pennsylvania. East North Central States: Ohio, Indiana, Illinois, Michigan, and Wisconsin. West North Central States: Minnesota, Iowa, Missouri, North Dakota, South Dakota, Kansas, and Nebraska. South Atlantic States: Delaware, Maryland, West Virginia, Virginia, North Carolina, South Carolina, Georgia, and Florida. East South Central States: Kentucky, Tennessee, Alabama, and Mississippi. West South Central States: Arkansas, Oklahoma, Louisiana, and Texas. Mountain States: Montana, Idaho, Wyoming, Utah, Colorado, New Mexico, Arizona, and Nevada. Pacific States: Washington, Oregon, and California.

[2] One abandoned branch extended between California and Nevada.

The smallest mileage abandoned was in the New England States, where only 54.47 miles were relinquished in 11 branches, of 4.95 miles average length. In the East North Central States there were 26 abandonments, involving 810.26 miles, or an average length of 31.16 miles; this is the largest mileage found in any of the sections, and is due in a large measure to the abandonment of eight whole lines in this section. The abandonment of the Indiana coal division of the Chicago & Eastern Illinois Railroad, comprising 162.1 miles, and the Chicago, Peoria & St. Louis Railroad in Illinois with its 234.3 miles of line, constituted a large portion of the abandoned mileage in this section.

In analyzing this situation the abandoned mileage in the various sections should be compared with the total railroad mileage. In this period, extending over almost five years, the 2,438 miles of abandoned railroads appear large when considered alone. If the trackage abandoned were brought together and placed end to end it would reach from New York City to Sacramento, Calif. In other words, it is equivalent to a transcontinental line of railroad, or, compared in another way, it is almost equivalent to the Wabash Railway system with its 2,476 miles of line. But when comparison is made with the total railroad mileage of the country these abandonments do not take on such a large aspect. On December 31, 1919, which was just prior to the passage of the law making it necessary for the railroads to file applications for the right to abandon lines with the Interstate Commerce Commission, the railroad mileage of the country was reported as 253,152 miles. For the country as a whole, therefore, the mileage abandoned since 1920 is less than 10 miles for every 1,000 miles in existence at the end of 1919.

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[1] Transportation act, 1920, sec. 1, par. 18.

[2] Finance Reports, Interstate Commerce Commission; vols. 65, 67, 70, 71, 72, 76, 79, 82, 86, 90, and 94.

In Table 3, in which the abandoned mileage is compared with the total railroad mileage as of December 31, 1919, for the various groups of States, it will be seen that the abandonments in the Middle Atlantic States were relatively the smallest, being only 4.1 miles per thousand, although the relative abandonments were approximately the same for the West North Central States. The greatest abandonment relative to the 1919 mileage took place in the East North Central States, where the abandoned mileage amounted to 18 miles for every thousand miles in existence in 1919.

Table 3.—*Comparison of abandoned railroad mileage with the total railroad mileage as of December 31, 1919*

Group of States	Total mileage of railroad, Dec. 31, 1919	Total mileage abandoned, 1920–1925	Abandoned mileage per 1,000 miles of road
	Miles	*Miles*	*Miles*
New England	7,985.59	54.47	6.8
Middle Atlantic	22,362.17	91.65	4.1
East North Central	44,868.75	810.26	18.0
West North Central	52,176.46	217.13	4.2
South Atlantic	32,434.56	248.60	7.6
East South Central	17,783.91	254.59	14.3
West South Central	32,972.64	353.90	10.7
Mountain	25,314.65	151.87	6.0
Pacific	17,253.44	256.95	14.8
Total	253,152.17	2,438.95	9.6

CAUSES OF SECTIONAL DIFFERENCES IN MILEAGE ABANDONED

The relative abandonments differ rather widely in the several groups of States because of the variations in the local conditions under which the railroad lines were built. In general it can be said that the relative abandonments in the States east of the Mississippi River were greater than in the States west of the Mississippi. For every thousand miles of railroad in the Eastern States 11.6 miles were abandoned and for every thousand miles west of the Mississippi only 7.6 miles were abandoned. The railroad mileages in these two broad sections of the country at the end of 1919 were almost the same; in the Eastern States there were 125,434.98 miles and in the Western States 127,717.19 miles. But the abandonments in the Eastern States were 1,459.47 miles, 49 per cent greater than the abandonment of 979.48 miles in the Western States. This difference can be largely explained by the fact that the railroads are older in the Eastern States, and a great many of the abandonments consisted of lines or branches of railroads built originally for the purpose of shipping to market various natural resources, the supply of which later became exhausted and no other traffic had been developed to warrant the continued operation. That type of railroad extension and development was not prevalent to the same extent in the Western States.

While the average length of the abandoned lines and branches has already been touched upon, it is of interest to determine how many were relatively short, and how many were more or less extensive. It will be readily seen from the classification in Table 4 that by far the largest number consisted of short stretches. Fifty-seven of the abandoned roads, constituting 47.5 per cent of the total number abandoned, were less than 10 miles long; those between 10 and 29 miles in length, 44 in number, were 36.6 per cent of the total. The remainder, or 19, ranging in length from 30 to 234 miles, constituted only 15.9 per cent of the abandonments. The lines abandoned are naturally of much greater average length than the branches.

Table 4.—*Classification of abandoned railroad lines and branches according to length, 1920–1925*

Length	Number of lines	Number of branches	Total
Miles			
0–4	----------	25	25
5–9	4	28	32
10–14	9	9	18
15–19	8	7	15
20–24	2	3	5
25–29	3	3	6
30–34	1	1	2
35–39	1	1	2
40–44	2	----------	2
50–54	4	1	5
55–59	----------	1	1
75–79	1	----------	1
90–94	2	----------	2
95–99	2	----------	2
160–164	1	----------	1
230–234	1	----------	1
Total	41	79	120

LOGGING AND MINING ROADS A MAJORITY OF THOSE ABANDONED

While the purposes for which these abandoned roads were built are not indicated in every case, it is clear that a majority were originally constructed as logging or mining roads. In some cases railroads already in operation built extensions or branch lines into certain sections or territories for the main purpose of tapping timber or mineral resources and in that manner sought to secure traffic consisting of the raw materials or products of such resources in their finished or semifinished state. In a great many cases new lines of railroad were built for this purpose and connected with lines or branches already in existence. The natural resources exhausted and traffic of other sorts having failed to develop, these lines and branches have been left without sufficient business to warrant continuance. That is the simple explanation of the majority of the abandonments.

In Table 5 the abandonments are classified according to the purpose of construction in order to show clearly the relative number of logging and mining roads.

The logging and mining roads, both lines and branches, aggregated 1,351.30 miles, or 55.4 per cent of the total abandoned mileage; and of these 1,351.30 miles 62.8 per cent were logging roads. Although the abandoned mileage of logging roads was greater than that of the abandoned mining roads, the mining roads were of greater average length. The lines built as logging roads and abandoned in this period averaged 25 miles in length; the lines built as mining roads averaged 53.3 miles. The same relative difference is noted in the average length of branch roads. The logging roads averaged 9.2 miles in length and the mining roads 16 miles. Of the 28 lines of logging and mining roads which were abandoned 24 were logging roads and only 4 mining roads. The tendency to build separate and individual lines of railroad to tap mineral resources was not as marked as the tendency to build such lines in order to market timber products.

TABLE 5.—*Railroad abandonments classified according to purpose of construction*

	Lines		Branches		Total	
	Number	Miles	Number	Miles	Number	Miles
Logging roads	24	600.66	27	248.59	51	849.25
Mining roads	4	213.20	18	288.85	22	502.05
Logging and mining roads	28	813.86	45	537.44	73	1,351.30
All others	13	795.77	34	291.88	47	1,087.65
Total	41	1,609.63	79	829.32	120	2,438.95

In most instances the lines of logging and mining railroads were projected and constructed by companies or interests engaged in the lumber or mining business and the railroad companies were in fact subsidiaries of such industrial companies. The branch lines abandoned were constructed in most cases by railroad companies which sought in this manner to obtain additional traffic and to meet the industrial and transportation needs of shippers.

MOTOR VEHICLE COMPETITION A MINOR CAUSE OF ABANDONMENT

The causes leading to the abandonment of this railroad mileage are in some instances simple and easily determined; in other instances the causes are more or less complicated. The lack of sufficient traffic to continue making the operation profitable or even possible is the fundamental reason for abandonment. The causes for the lack of traffic are varied. The primary causes cited are: (1) Exhaustion of natural resources; (2) competition of other railroads; (3) competition of motor vehicles operating on highways; (4) rearragement of lines of railroad; and (5) miscellaneous causes. The reasons for the abandonments, grouped according to this classification, are summarized in Table 6.

TABLE 6.—*Primary causes of lack of traffic*

Cause	Number of railroads	Percentage of number	Length	Percentage of length
		Per cent	*Miles*	*Per cent*
Exhaustion of natural resources	78	65.0	1,411.20	57.8
Competition of other railroads	14	11.7	713.34	29.3
Competition of motor vehicles	10	8.4	104.46	4.3
Rearrangement of lines of railroad	5	4.1	32.64	1.3
Miscellaneous	13	10.8	177.31	7.3
Total	120	100.0	2,438.95	100.0

The outstanding fact deduced from this analysis and summary is that the exhaustion of the natural resources which furnished the bulk of the traffic for these abandoned lines and branches of railroads accounted for 65 per cent of the number of abandonments and for 57.8 per cent of the abandoned mileage. This one factor was the principal cause of the abandonment of considerably more than half of the total abandoned mileage. Competition of other railroads resulted in traffic losses which brought about the abandonment of almost 30 per cent of the total abandoned mileage, and it is of special interest to note that less than 5 per cent of the abandoned mileage can be attributed to motor-vehicle competition as the primary cause.

REASONS FOR ABANDONMENT ILLUSTRATED

To illustrate these primary causes typical cases are cited. The Alabama & Mississippi Railroad Co. was authorized October 21, 1921, to abandon its line from Vinegar Bend, Ala., to Pascagoula, Miss., a distance of 75.5 miles.[3] The railroad was built to market timber products. At the time of abandonment the territory traversed consisted largely of cut-over timber lands. The agricultural development was so small that only three carloads of agricultural products originated on the line in 1920. The passenger traffic was negligible. For five years prior to 1920 the line incurred net operating deficits. The principal communities on the line were served by other railroads and suffered no hardship as a result of the abandonment. The road was in the hands of a receiver and no objection was made to the proposed plan of abandonment.

For similar reasons the Pere Marquette Railway Co. was authorized October 21, 1921, to abandon a branch line from Harrison to Leota, Mich., 9.8 miles in length.[4] It had been built in 1897 as a logging road. The timber had become exhausted and the country tributary to the branch was sparsely populated. No cities or towns had grown up, and after the lumber was all cut and moved the traffic came practically to a standstill, making the operation of the branch line no longer necessary. No opposition to abandonment was encountered. This is an illustration of what happened in most cases where branch roads, built primarily to take care of the transportation of certain natural resources, were abandoned when these resources became exhausted and no other kind of traffic had developed in amount sufficient to keep up railroad operation.

An example of the abandonment of a line of railroad brought about primarily by the competition of other railroads is the case of the Chicago, Peoria & St. Louis Railroad, extending from Pekin to East St. Louis, a distance of 234.30 miles, through one of the richest agricultural sections of Illinois.[5] This was the longest section of railroad abandoned in the five-year period. It passed through 35 cities and villages, 20 of which had no other rail transportation facilities. It was an old line of railroad, a part of which was constructed prior to the Civil War. It had passed through several receiverships and was in the hands of a receiver again at the time the abandonment proceedings were brought.

The principal reason advanced for the decrease in traffic was the loss of freight tonnage to other railroads brought about through short routing. The local business was not sufficient to maintain the road in operation and the connecting railroads were not willing to divert any of their through traffic voluntarily. During the war the road's freight-soliciting agencies in outside cities were done away with, and when the road was returned to its owners the business formerly secured in that manner could not again be obtained. The lack of earnings also resulted in undermaintenance of road bed and rolling stock, so that the service became poorer and poorer. Motor-vehicle competition was also cited as one of the reasons for a loss of a part of the local traffic. Highway transportation could not, however, have been regarded as the primary cause leading to abandonment. The highways in the territory were greatly improved and these improved highways made it possible for communities dependent on this line of railroad for service to have their transportation needs satisfied without any undue hardships. The average distance to other railroads from the 20 points which were not on another line was only 6.6 miles. The main reason leading to abandonment was that the railroad was located in too highly competitive railroad territory. The lack of sufficient traffic was evident even prior to the time when competition of the highways became pronounced.

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[3] 70 I. C. C. 531.

[4] 70 I. C. C. 535.

[5] 76 I. C. C. 801.

ABANDONMENTS DUE TO HIGHWAY COMPETITION CITED

The abandonment of the Ocean Shore Railroad in California was caused primarily by highway competition, according to the commission's report.[6] The road extended from San Francisco to Tunitas Glen, a distance of 53.6 miles. It was constructed in 1909, but in recent years the traffic had diminished steadily on account of the increasing competition of motor vehicles. Before the road was in operation two years, receivership proceedings were had. According to the evidence in the case, the gross revenues never exceeded the operating expenses. In this case it is a question whether the railroad could have been successfully operated even if highway transportation had not entered as a competitive factor.

The Boston & Maine Railroad Co. was authorized to abandon several of its branch lines leading from the main line to summer resorts.[7] The branch extending to the Profile House, N. H., 9.11 miles in length, was allowed to be abandoned because automobile traffic had reduced the passenger traffic to almost nothing. This was brought about through the construction of a good highway from the Profile House to stations on the company's main line. The year's revenue was given at $1,713 and the operating expenses $11,227. It is questionable whether this section of railroad, operated only during the summer months, ever yielded revenues in excess of operating expenses.

The construction of an improved highway the length of Barnegat Island, N. J., made it possible for the Pennsylvania Railroad Co. to discontinue operating its branch line, 8.12 miles long, on that island.[8] The highway paralleled the whole length of the railroad. Trucks and automobiles meet all the traffic demands. The record shows that the railroad was operated at a loss continually since 1894, when it was constructed. When the improved highway took so much of the traffic that there was scarcely any left for the railroad to carry, it also made it possible for the railroad company to be relieved of a financial burden which had been a drain upon the net revenues of the system as a whole. The public appeared to be satisfied with the abandonment, since there was no strenuous opposition to the discontinuance of service from any source at the time of the hearing.

In a few cases abandonments were brought about through rearrangements in line or through new construction which resulted in the shortening of routes, making the old stretches of railroad no longer necessary. An example was the abandonment of a branch of the Delaware & Hudson Co. in New York which formerly was a part of its main line. A short line was built which made this section of the road a branch, with no through traffic to carry.[9] Improved highways have been constructed by which the public's transportation needs are adequately met, and it was thus made possible to abandon this branch without interfering with the public convenience and necessity. There were five abandonments of this character.

A number of abandonments were authorized where it was shown that there was not sufficient traffic, though no definite reasons were given for traffic deficiencies or decreases. A case in point was the abandonment of the Orangeburg Railway, a line 17.7 miles long, extending from Orangeburg to North, S. C.[10] The road, built in 1913, was originally planned to extend to Charleston. It never earned its fixed charges. As early as 1916 it was already in the hands of a receiver. There was no evidence that traffic was taken away from it by other railroads or by highways. The plain facts were that there never was enough business for a railroad in the territory and that it should not have been built in the first place.

Another example of an abandonment of this character was that of the branch of the Chicago, Milwaukee & St. Paul Railway extending from Cogswell to Harlem, N. Dak., a distance of 5 miles.[11] Harlem at one time had two general stores and two grain elevators. The stores have been discontinued; one of the elevators was dismantled, and the other was destroyed in a windstorm and was not rebuilt. The farmers operating in the territory traversed by this branch do their business in the towns on the main line of the railroad. Thus it was no longer necessary to continue the operation of the branch, because nobody's convenience and necessity was served by it.

THE RIGHT TO ABANDON REFUSED IN SOME INSTANCES

Not every application for abandonment was granted by the commission. One line of railroad with a mileage of 40.80 miles and 8 branches, involving a mileage of 75.76 miles, was denied the authority to abandon. In the case of the Frankfort & Cincinnati Railway, which desired to abandon its line from Frankfort to Paris, Ky., the commission found that, while there had been a loss in traffic due to the closing down of distilleries and the use of motor vehicles on the highways, it seemed possible for the company, by making additional efforts, to get more through business and that a large number of people were dependent on the railroad because of poor roads in certain sections served by it.[12] It was further brought out that the Louisville & Nashville Railroad Co. owned the majority of the stock of this road, and that it could therefore be regarded as that road's subsidiary.

The other denials all involved large railroad companies which desired to abandon certain branch roads because of their unprofitableness.[13] In denying authority to abandon these branches the commission invariably found that the branches were necessary to meet the present and future transportation demands of the communities affected and that, while operating deficits were shown, "a railroad company could not claim the right to earn a net profit from every section into which its road might be divided.

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[6] 67 I. C. C. 760.

[7] 70 I. C. C. 266.

[8] 79 I. C. C. 506.

[9] 94 I. C. C. 376.

[10] 67 I. C. C. 789.

[11] 82 I. C. C. 274.

[12] 86 I. C. C. 740.

[13] 71 I. C. C. 225; 72 I. C. C. 25; 72 I. C. C. 267; 72 I. C. C. 303; 79 I. C. C. 431; 86 I. C. C. 637; 86 I. C. C. 609; 94 I. C. C. 624.

The jurisdiction and power of the Interstate Commerce Commission to grant authority to a railroad to abandon a line of railroad or a part thereof was challenged in several instances by State authorities. After the commission had issued its order[14] authorizing the abandonment of the line owned and operated by the Eastern Texas Railroad Co., the State of Texas brought a suit in the United States district court against the commission, the railroad company, and others in which the annulment of the commission's order was sought. The district court decided against the State, whereupon the State appealed the case to the United States Supreme Court. The Eastern Texas Railroad was wholly an intrastate road, and it was claimed that paragraph 18, section 402 of the transportation act did not give the Interstate Commerce Commission the power to authorize the abandonment of a railroad with respect to intrastate commerce. The Supreme Court in reversing the order of the lower court held[15] that the continued operation of the road in intrastate service was of local concern and that any shortage of earnings would not affect or place a burden upon interstate commerce because no carrier engaged in interstate commerce would be required to make up any deficits. The court interpreted this section of the transportation act to mean that the commission did not have the authority to grant abandonment certificates so far as purely intrastate business was concerned and where neither interstate nor foreign commerce was burdened thereby.

Whether the railroad in this case was entitled to discontinue its intrastate business was not considered by the court in that particular proceeding. This question, however, had to be decided by the court within a period just long enough for another appealed case to be brought to the court's attention.

RIGHT TO ABANDON UNPROFITABLE INTRASTATE OPERATIONS AFFIRMED

The Eastern Texas Railroad having been authorized to abandon its interstate traffic now proceeded to discontinue its intrastate business as well as its interstate business. Two injunction suits were filed forthwith in the United States district court; in one the State of Texas asked for an injunction against the railroad company to prevent it from dismantling and abandoning its railroad; in the second suit an injunction was sought by the railroad company to prevent the State authorities from interfering with the company's abandonment plans. The district court granted the injunction applied for by the railroad company and denied the State's request. In the appeal to the United States Supreme Court it was there argued by the State, as in the lower court, that the company's charter required it to operate for a period of 25 years, which had not yet expired, and that it was liable for damages if it failed to continue operating. A law passed by the Texas Legislature which required "all railroads within the State to operate at least one passenger train a day" was also cited as being a part of the company's charter requirements, even though passed subsequent to the granting of the charter. In the testimony in the district court it was shown that the operating costs were $84,000 a year and that the estimated revenues would not be in excess of $50,000, of which not more than $20,000 would be derived from intrastate business. The company maintained that a requirement to operate would, in the light of these facts, deprive it of its property without due process of law and in violation of the provisions of the fourteenth amendment. The Supreme Court affirmed the action of the lower court and held that the charter was in the nature of a permit and did not require its holder to operate at a loss nor could the "train-a-day" law be considered as more than one of the regulations of the State under its police powers.[16]

After the first of these Texas Railroad Co. decisions by the Supreme Court the Interstate Commerce Commission, in granting certificates authorizing the abandonment of lines of railroad situated wholly within a State, always limited its orders to the discontinuance of operations with respect to interstate commerce.

In opposing the application of the Southern Pacific Co. for authority to abandon one of its branches located wholly in Nevada the authorities of that State claimed that the Interstate Commerce Commission was without jurisdiction because of the intrastate character of the branch.[17] It was shown that the branch was operated at a loss and that there was no great demand for its service. In that case the commission held that even though the branch was located wholly within the State it constituted a part of the Southern Pacific Co.'s interstate system of transportation and that the losses incurred on this branch would be reflected in the accounts of the company and might in that manner be regarded as a possible burden upon interstate commerce.

Although, as has been shown, the Interstate Commerce Commission has authorized the abandonment of a large railroad mileage in the last five years, it should be borne in mind also that during this same period[18] certificates of public convenience and necessity were issued by the commission authorizing the new construction of 2,673.36 miles of railroad. This involved the construction of 17 lines aggregating 1,159.52 miles and 67 branches aggregating 1,513.84 miles. How many miles were actually constructed can not be determined from the official reports, but it is reasonable to assume that a substantial portion of this mileage was built and put into operation.

It is generally believed that highway transportation will be considered an important element in the future railroad development of the country. Highway transportation may make it necessary to make railroad extensions into certain territories and it will also make it possible to relinquish the operation of certain low-traffic lines and branches, thus saving money for the railroads and providing the public with reasonably adequate transportation service.

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[14] 65 I. C. C. 436.

[15] 258 U. S. 204.

[16] 264 U. S. 79; 283 Fed. 584.

[17] 72 I. C. C. 404.

[18] Nov. 1, 1920, to May 1, 1925.

THE ACTION OF SULPHATE WATER ON CONCRETE[1]

A SUMMARY OF TESTS OF SPECIMENS IMMERSED FOR ONE YEAR IN MEDICINE LAKE, S. DAK.

Reported by **DALTON G. MILLER**, Drainage Engineer, United States Bureau of Public Roads

THE field work of 1919 and 1920[2] resolved the so-called concrete-alkali problem of Minnesota, as related to the use of concrete drain tile into a problem of the effect of the sulphates of magnesium ($MgSO_4$) and sodium (Na_2SO_4). Subsequent to this work a cooperative laboratory was established at University Farm at St. Paul, Minn., in 1921, and experiments have been there conducted by the department of agriculture of the University of Minnesota, the State Department of Drainage and Waters, and the United States Bureau of Public Roads. These laboratory experiments were designed principally to aid in the general improvement of farm drain tile and particularly to develop tile that will endure under the wide range of soil conditions peculiar to Minnesota. The results are applicable, however, to many other sections of the United States and to concrete culvert pipe subjected to the action of sulphate waters under conditions similar to those to which drain tile are subjected.

At the cooperative laboratory several hundred 2 by 4 inch concrete and mortar cylinders of various types were made and stored in earthenware jars containing solutions of the two sulphate salts. Much has been learned from this work and certain conclusions seem to be warranted. It is recognized, however, that any laboratory results to be of value must be supported by tests under field conditions, and it was with this in view that the cylinders upon which this report is based were installed in Medicine Lake, S. Dak., 17 miles northwest of Watertown. In choosing a suitable site for field installations this lake was selected as being very nearly ideal for the purpose in the essentials of kinds and quantities of salts and, also, as being reasonably accessible.

ANALYSES OF MEDICINE LAKE WATER

Analyses of water samples taken from Medicine Lake at different seasons of the year are given in Table 1. As indicated, the total salt content has ranged between 2.34 and 4.72 per cent, consisting almost entirely of a combination of magnesium and sodium sulphates in which the magnesium salt greatly predominates. The larger of these two figures represents water with a total salt content slightly in excess of any soil water so far found in contact with any drain tile in Minnesota and may safely be assumed to be as severe as is ordinarily encountered in agricultural lands anywhere.

As in the previous laboratory tests the specimens used were 2 by 4 inch cylinders, the majority of concrete, but a number also of Ottawa sand mortar. As suggested by the previous laboratory work, in which it was found that proper curing is one of the most essential factors in developing resistance to the action of sulphate waters, the samples installed in the lake represent a wide range of curing conditions, including storage for varying periods in the moist closet, in steam, in water vapor at various temperatures, in tap water and distilled water, and in air, and various combinations of the several treatments.

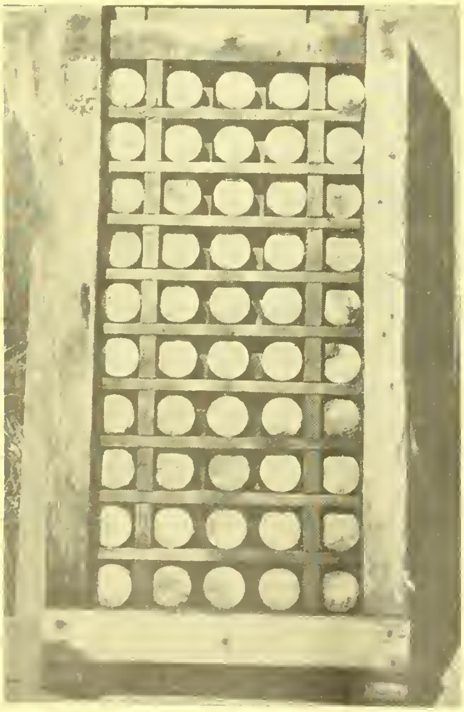

FIG. 1.—Cylinders from series 77 to 81, cured in water vapor at 212° F.

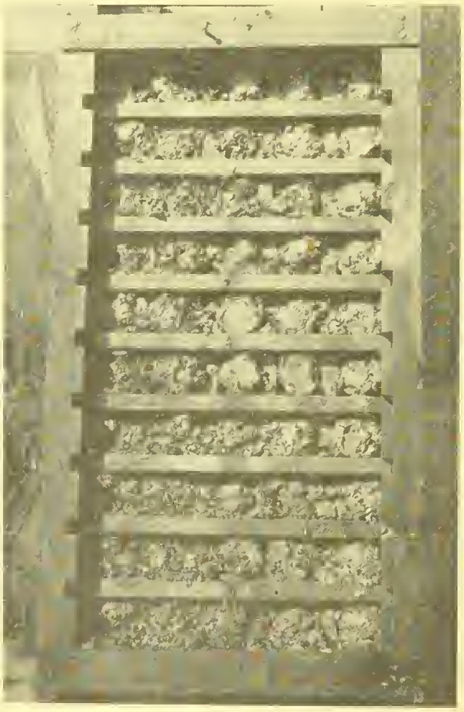

FIG. 2.—Cylinders from series 82 to 86, cured in water vapor at 155° F.

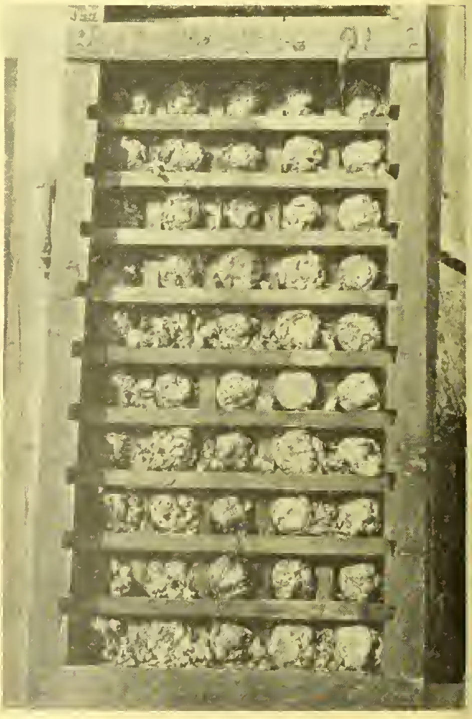

FIG. 3.—Cylinders from series 87 to 91, cured in water vapor at 100° F.

Portland cement concrete cylinders after 18 months in Medicine Lake, S. Dak. Cylinders in each crate were identical except for curing

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[1] University of Minnesota Paper No. 562, Journal Series.

[2] Report of concrete-alkali investigations in Minnesota, 1919–20. Department of Drainage and Waters, State of Minnesota.

Table 1.—*Analyses of water from Medicine Lake, S. Dak.*

(Analyses by the Water and Beverage Laboratory, Bureau of Chemistry, U. S. Department of Agriculture)

Sample No.	Date taken	Radicals								
		Na (Calc.)	Ca	Mg	NO_3	Cl	SO_4	CO_3	HCO_3	Total
		Milligrams per liter (parts per 1,000,000)								
778.2	Dec. 10, 1923	3,755	1,006	5,012	1	580	28,898	84	282	39,618
781.2	Feb. 14, 1924	4,344	1,082	6,116	Trace.	596	34,533	54	490	47,215
803.2	Apr. 29, 1924	395	344	4,276	Trace.	396	17,585	98	323	23,417
808.2	July 1, 1925	3,649	436	4,910	Trace.	464	27,067	114	158	36,828
		Percentage reacting values								
778.2		13.05	4.01	32.94	0.00	1.31	48.10	0.22	0.37	100.00
781.2		12.67	3.62	33.71	.00	1.13	48.22	.12	.53	100.00
803.2		2.23	2.22	45.55	1.00	.45	47.44	.42	.69	100.00
808.2		13.58	1.86	34.56	.00	1.12	48.25	.41	.22	100.00

Besides varying the curing treatment several different cements were tried including three high-alumina brands, and a number of the specimens were designed to test the effect of various admixtures and impregnations. The number of types of cylinders installed in the lake has purposely been kept to the minimum, keeping in mind as the principal purpose the development of a concrete suitable for use in the commercial manufacture of ordinary farm drain tile, and following the lines indicated as most hopeful by the laboratory experiments. Because of these conditions, the 2 by 4 inch concrete cylinders referred to as standard laboratory cylinders, which constitute the majority of the specimens, were made with a well-graded aggregate the screen size of which was less than three-eighths inch. Also, in nearly all cases, the mix was 1:3 and the relative consistency 1. A 1:3 mix is as rich as is ordinarily used for drain tile, and it has been found by the laboratory work that a leaner mix than this is very much less resistant. A richer mix, while more resistant, is rarely used, as the cost of such tile generally is prohibitive. The relative consistency of 1 is as wet as can be used in machine-molded tile, and it has been found that consistencies greatly varying from this are considerably less resistant.

In addition to these standard laboratory cylinders of concrete a number of mortar cylinders made with standard Ottawa sand were also placed in the lake with the idea that they would fairly well represent the poorly graded aggregate that is too often used in the smaller sizes of tile.

HIGH-ALUMINA AND STEAM-CURED CYLINDERS BEST AFTER ONE YEAR IN LAKE

All data with respect to each series of specimens are given in Tables 2 and 3. No effort will be made at this time to analyze in detail all the results of the examinations and tests of the cylinders which have now been stored in the lake for one year, as the exposure period is too short to warrant the drawing of definite conclusions with respect to many of the series. From even a cursory study of the data, however, certain significant facts are evident and these may be briefly summarized as follows:

1. None of the high-alumina cement cylinders showed evidence of any deleterious action. One hundred and twenty-five of such cylinders in 25 series, variously cured and stored in the lake for one year, were tested for compression, together with the same number of check cylinders stored for the same period in the laboratory tank. In 21 series the strength ratio of the cylinders from the lake in comparison with those from the tank exceeded 100 per cent, in 1 series it was 99 per cent, in 2 it was 98 per cent, and in 1 it was 94 per cent. The maximum for any series was 144 per cent, while the average for the 25 series was 115 per cent. It is possible that the 15 per cent excess strength of the cylinders from Medicine Lake is due, at least in part, to the generally lower temperature of the lake water as compared with that of the laboratory tank. It is known that the reaction to temperature of high-alumina cement is considerably different from that of standard Portland cement, as the time required for both the initial and final set of the high-alumina brands is materially shortened by reduction, within certain limits, of the temperature of the curing medium, which is contrary to the action of standard Portland cement. It is also possible that the sulphates of the lake water may have had some accelerating action.

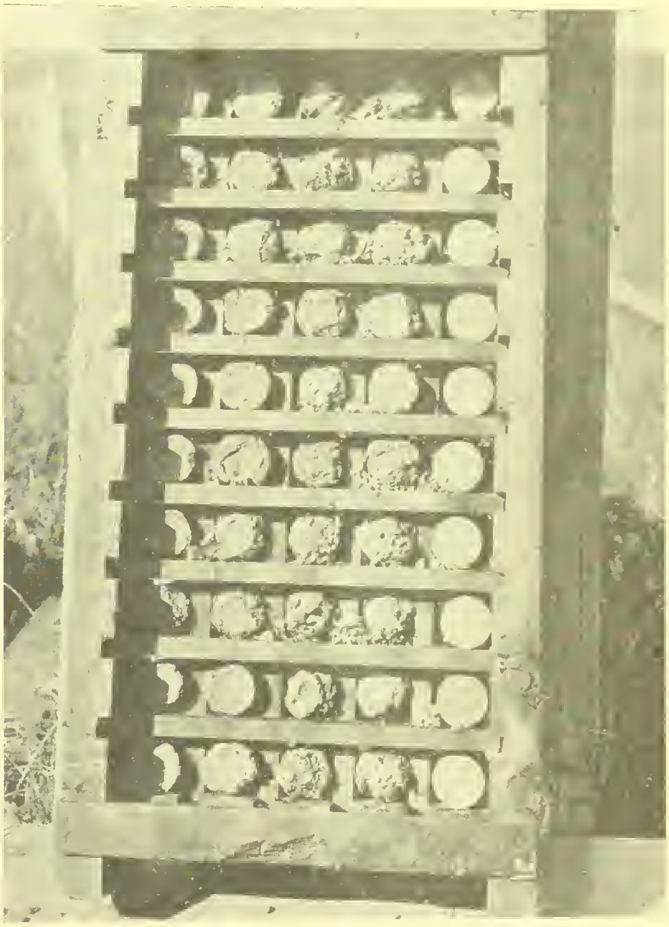

Fig. 4.—From left to right, after 24 hours in the moist closet, cylinders in each row were cured as follows:

Series 92.—Tap water 27 days, air 28 days
Series 93.—Moist closet 2 days, air 25 days
Series 94.—Water vapor at 100° F. 2 days, air 25 days
Series 95.—Water vapor at 155° F. 2 days, air 25 days
Series 96.—Water vapor at 212° F. 2 days, air 25 days

Portland cement concrete cylinders after 18 months in Medicine Lake, S. Dak. Cylinders were identical except for the curing conditions.

2. None of the standard Portland cement cylinders cured in steam at a temperature of 212° F., regardless of the length of time they were so cured or of other variables, showed any surface action, those from the lake looking fully as good at one year as the ones from the laboratory tank. Ninety-five cylinders from 19 series after one year in the lake had an average strength

TABLE 2.—*Description and tests of all laboratory standard concrete cylinders*

(Fineness modulus of aggregate for all cylinders 4.67. Mix 1 : 3 except as noted for 4 series)

Series No.	Cement laboratory No.[1]	Cement	Admixture or impregnation	Curing method						Absorption at 21 days	Average compression tests							Strength ratio	Length increase	
				Time in moist closet	Time in steam at 212° F.	Time in water vapor	Temperature of water vapor	Time in water	Time in air		7 days	28 days	1 year						Number	Increase
													Tank specimens		Lake specimens					
													Number	Strength	Number	Strength				
				Hours	*Hours*	*Hours*	*° F.*	*Days*	*Days*	*P. ct.*	*Lbs.*	*Lbs.*		*Pounds*		*Pounds*		*Per cent*		*Inch*
77	11.4	One-half Northwestern, one-half Universal.		3	69				25	7.95	1,110	1,250	5	1,250	5	1,210		97	10	+0.0006
78	11.4	do		6	66				25	7.18	2,260	3,040	5	3,130	5	2,840		91	10	-.0008
79	11.4	do		12	60				25	6.86	3,400	4,120	5	4,550	5	4,410		97	10	-.0015
80	11.4	do		24	48				25	6.94	3,060	4,020	5	4,600	5	4,330		94	10	-.0015
81	11.4	do		48	24				25	6.68	2,910	3,850	5	4,630	5	4,410		95	10	-.0016
82	11.4	do		3		69	155		25	7.0	3,365	4,210	5	5,380	5	500		9		
83	11.4	do		6		66	155		25	6.9	3,840	4,720	5	5,700	5	490		9		
84	11.4	do		12		60	155		25	6.7	4,440	4,960	5	5,250	5	1,030		20		
85	11.4	do		24		48	155		25	6.6	4,355	4,550	5	6,460	5	1,220		19		
86	11.4	do		48		24	155		25	6.7	3,545	4,210	5	6,250	5	1,660		27		
87	11.4	do		3		69	100		25	6.5	3,730	4,120	5	5,110	5	1,430		28		
88	11.4	do		6		66	100		25	6.7	3,590	4,320	5	4,960	5	1,370		28		
89	11.4	do		12		60	100		25	6.7	4,050	4,680	5	5,270	5	1,290		25		
90	11.4	do		24		48	100		25	6.7	4,080	4,220	5	5,240	5	1,630		31		
91	11.4	do		48		24	100		25	6.8	3,510	4,060	5	5,010	5	1,920		38		
92	11.4	do		24				[2] 27	28	6.7	2,620	4,540	5	5,690	5	3,780		66	10	-.0001
93	11.4	do		72					25	7.1	3,380	3,560	5	5,120	5	3,150		62	9	+.0014
94	11.4	do		24		48	100		25	6.8	3,810	4,000	5	5,640	5	2,020		36	9	+.0021
95	11.4	do		24		48	155		25	6.4	4,240	4,370	5	5,510	5	1,660		30	7	+.0059
96	11.4	do		24	48				25	6.5	3,770	4,410	5	4,560	5	4,300		94	10	-.0004
97	11.4-8.4	do		24		48	155		25	6.3	3,900	4,770	5	5,690	5	1,880		33	7	+.0031
98	11.4-8.4	do		24	24	24	155		25	6.6	4,180	4,120	5	5,020	5	4,340		87	10	-.0004
99	11.4-8.4	do		24		72	155		24	6.4	4,490	4,830	5	5,410	5	1,530		28	10	+.0085
100	11.4-8.4	do		24	24	48	155		24	6.2	4,310	4,960	5	5,730	5	5,540		97	10	-.0001
101	11.4-8.4	do		24	48	24	155		24	6.4	4,110	4,710	5	4,890	5	4,640		95	10	-.0007
102	17.4	do		24		48	155		25	6.2	4,550	4,730	5	5,580	5	980		18	10	+.0501
103	17.4	do		24	24	24	155		25	6.0	4,600	5,510	5	5,290	5	4,980		94	10	+.0004
104	17.4	do		24		72	155		24	6.1	4,910	5,180	5	5,180	5	1,270		25	10	+.0162
105	17.4	do		24	24	48	155		24	6.0	4,820	5,070	5	4,910	5	5,220		106	10	+.0001
106	17.4	do		24	48	24	155		24	5.7	5,110	5,900	5	5,320	5	5,250		99	10	-.0002
137	14.4	Ciment electrique.[3]		24		48	155		25	6.25	3,250	3,550	5	2,830	5	3,400		120	10	-.0006
138	14.4	do		24	24	24	155		25	6.15	3,150	3,670	5	2,860	5	3,320		116	10	-.0004
139	14.4	do		24		72	155		24	6.09	3,160	3,840	5	2,640	5	3,790		144	10	-.0010
140	14.4	do		24	24	48	155		24	6.13	3,190	3,890	5	2,730	5	3,790		139	10	-.0007
141	14.4	do		24	48	24	155		24	6.09	3,360	3,800	5	3,020	5	3,670		121	10	-.0006
147	14.4	do		24				[4] 27		6.25	4,560	4,890	5	4,560	5	5,220		114	10	-.0005
148	14.4	do		72					25	7.00	5,020	4,810	5	5,310	5	5,960		112	10	+.0011
149	14.4	do		24		48	100		25	6.55	5,370	5,510	5	5,240	5	5,450		104	10	+.0015
150	14.4	do		24		48	155		25	6.16	3,360	3,870			5	3,400			10	-.0014
151	14.4	do		24	48				25	6.07	3,300	3,750	5	2,840	5	3,650		129	10	-.0011
152	14.4	do		24				[4] 20	343	6.35	4,570	5,040	5	5,100	5	5,040		99		
153	14.4	do		24				[4] 20	56	6.49	4,830	4,910	5	5,120	5	6,280		123	10	-.0001
154	14.4	do		24				[4] 20	28	6.51	4,760	4,850	5	5,450	5	5,620		103	10	+.0005
155	14.4	do		24				[4] 20	14	6.54	4,250	4,960	5	4,570	5	5,770		126	10	+.0004
156	14.4	do		24				[4] 20		6.34	4,710	4,860	5	4,380	5	4,960		113	10	-.0005
157	18.4	One-half Northwestern, one-half Universal.	Blast furnace slag admixed.	24				[4] 20		6.52	2,420	3,960	5	6,030	5	4,230		70	10	+.0003
158	18.4	do	do	24				[4] 20	35	5.97	2,880	4,180	5	5,440	5	5,170		95	10	-.0008
159	18.4	do	do	24				[4] 20	35	6.66	2,410	4,190	5	5,670	5	5,560		98	10	-.0011
160	18.4	do	do	24		48	155		35	6.38	3,850	4,010	5	5,300	5	4,800		91	10	+.0004
161	18.4	do	do	24		48	155		35	6.85	3,800	4,020	5	5,450	5	4,830		89	10	+.0001
162	18.4	do	Calcium chloride admixed.	24				[4] 20		5.72	2,870	4,330	5	5,530	5	3,910		71	10	+.0020
163	18.4	do	do	24				[4] 20	35	7.06	2,200	3,010	5	4,280	5	3,550		83	10	+.0020
164	18.4	do	do	24				[4] 20	35	5.94	2,940	4,000	5	5,450	5	5,050		93	10	-.0003
165	18.4	do	do	24		48	155		35	7.09	3,800	3,640	5	4,600	5	3,300		72	10	+.0042
166	18.4	do	do	24		48	155		35	5.82	4,840	4,920	5	4,990	5	4,460		89	10	+.0015
167	18.4	do	Cal admixed	24				[4] 20		6.02	3,200	4,150	5	4,860	5	3,910		80	10	+.0018
168	18.4	do	do	24				[4] 20	35	5.84	3,180	4,290	5	5,570	5	4,620		83	10	+.0001
169	18.4	do	do	24				[4] 20	35	5.84	3,250	4,280	5	4,550	5	4,800		105	10	-.0006
170	18.4	do	do	24		48	155		35	5.96	4,990	4,670	5	5,030	5	5,010		100	10	+.0008
171	18.4	do	do	24		48	155		35	5.97	4,550	4,690	5	5,140	5	4,980		97	10	+.0015
172	18.4	do		24				[4] 20	35	5.57	2,870	4,190	5	5,300	5	4,370		82	9	-.0001
173	18.4	do	Ironite admixed	24				[4] 20	35	5.87	2,950	3,980	5	4,890	5	5,020		102	9	-.0004
174	18.4	do		24		48	155		35	6.11	4,020	4,390	5	4,330	5	3,910		90	10	+.0004
175	18.4	do	Ironite admixed	24		48	155		35	6.06	3,880	4,440	5	5,510	5	4,630		84	10	+.0006
176	18.4	do	do	24		48	100		35	6.08	3,680	4,220	5	5,250	5	5,190		99	10	+.0003
177	18.4	do		24				[4] 20	35	5.91	2,520	4,200	5	4,930	5	3,670		74	10	-.0010
178	18.4	do	Volcanic ash admixed.	24				[4] 20	35	6.79	1,900	3,210	5	5,450	5	2,960		54	10	-.0015
179	18.4	do		24		48	155		35	5.83	4,110	4,510	5	5,210	5	3,820		73	10	+.0009
180	18.4	do	Volcanic ash admixed.	24		48	155		35	7.59	3,240	3,340	5	4,440	5	3,600		81	10	-.0003
181	18.4	do	do	24		48	100		35	7.28	2,710	2,820	5	5,130	5	4,290		84	10	-.0001
182	18.4	do		24				[4] 20	35	5.98	2,802	4,010	5	5,780	5	4,450		77	10	-.0005
183	18.45	do	Alkagel "A" admixed.	24				[4] 20	35	5.31	2,260	3,560	5	4,570	5	3,520		77	10	-.0011
184	18.4	do		24		48	155		35	5.96	4,070	4,530	5	5,100	5	4,240		83	10	+.0002
185	18.4	do	Alkagel "A" admixed.	24		48	155		35	4.97	3,280	3,940	5	4,570	5	3,300		72	10	-.0001
186	18.4	do	do	24		48	100		35	5.43	3,120	3,730	5	3,820	5	3,550		93	10	-.0006
208	25.4	Atlas lumnite [3]		24				[4] 20	35	4,87	6,360	6,500	5	6,260	5	7,420		118	10	-.0007
209	25.4	do		24				[4] 20	35	4.96	6,310	6,470								
210	25.4	do		24				[4] 20	35	4.94	7,200	7,100								
211	25.4	do		24				[4] 20	35	4.95	7,440	6,970								
212	25.4	do		24				[4] 20	35	4.91	6,180	6,870								
243	28.4	Super cement [5]		24				[4] 20	35	6.15	2,840	3,800	5	5,400	5	4,020		74	10	-.0015
244	28.4	do		24				[4] 20	35	5.96	2,900	3,740	5	5,300	5	4,100		77	10	-.0027
245	28.4	do		24				[4] 20	35	6.23	2,768	3,730	5	5,250	5	3,600		69	10	-.0006
246	28.4	do		24				[4] 20	35	6.25	2,980	3,690	5	5,750	5	4,250		74	10	-.0019

TABLE 2.—*Description and tests of all laboratory standard concrete cylinders*—Continued

Series No.	Cement laboratory No.[1]	Cement	Admixture or impregnation	Curing method						Absorption at 21 days	Average compression tests							Length increase	
				Time in moist closet	Time in steam at 212° F.	Time in water vapor	Temperature of water vapor	Time in water	Time in air		7 days	28 days	1 year				Strength ratio	Number	Increase
													Tank specimens		Lake specimens				
													Number	Strength	Number	Strength			
				Hours	*Hours*	*Hours*	° *F.*	*Days*	*Days*	*P. ct.*	*Lbs.*	*Lbs.*		*Pounds*		*Pounds*	*Per cent*		*Inch*
247	28.4	Super cement		24				[4] 20	35	6.31	3.070	3,880	5	5,130	5	4,170	81	10	−0.0008
252	58.4	Ciment fondu [3]		24				[4] 20	35	5.40	7,410	7,250							
253	58.4	do		24				[4] 20	35	5.40	8,040	7,200							
256	61.4	One-half Northwestern, one-half Universal.		24				[4] 20	35	5.84	2,780	3,950							
257	63.4	Northwestern		24				[4] 20	35	5.84	2,720	4,290							
258	62.4	Universal		24				[4] 20	35	5.78	2,960	3,790							
259	33.4	Lehigh		24				[4] 20	35	5.41	3,340	4,780							
260	34.4	Hawkeye		24				[4] 20	35	6.14	3,340	4,540							
[6] 286	27.4	Atlas lumnite [3]		24				[4] 20	35										
287	27.4	do		24				[4] 20	35										
288	27.4	do		24				[4] 20	35										
[7] 289	27.4	do		24				[4] 20	35										
[8] 290	27.4	do		24				[4] 20	35										
[8] 291	58.4	Ciment fondu [3]		24				[4] 20	35										

[1] For tests of cement see Table 5

[2] Tap water.

[3] High-alumina cement.

[4] Distilled water.

[5] Special cement.

[6] Laboratory standard cylinder except in mix, which is 1:2.

[7] Laboratory standard cylinder except in mix, which is 1:4.

[8] Laboratory standard cylinder except in mix, which is 1:5.

TABLE 3.—*Description and tests of all standard Ottawa sand mortar cylinders*

(Fineness modulus of aggregate for all cylinders 3. Mix 1 : 3 except as noted for 4 series)

Series No.	Cement laboratory No.[1]	Cement	Admixture or impregnation	Curing method						Absorption at 21 days	Average compression tests							Length increase	
				Time in moist closet	Time in steam at 212° F.	Time in water vapor	Temperature of water vapor	Time in water	Time in air		7 days	28 days	1 year				Strength ratio	Number	Increase
													Tank specimens		Lake specimens				
													Number	Strength	Number	Strength			
				Hours	*Hours*	*Hours*	° *F.*	*Days*	*Days*	*P. ct.*	*Lbs.*	*Lbs.*		*Pounds*		*Pounds*	*Per cent*		*Inch*
107	17.4	One-half Northwestern, one-half Universal.		24		48	155		25	9.8	2,660	3,080	5	3,580	4	960	27	8	+0.0126
108	17.4	do		24	24	24	155		25	9.8	2,590	2,910	5	2,930	5	2,540	87	10	+.0002
109	17.4	do		24		72	155		24	9.6	2,670	3,220	5	3,490	5	930	27	10	+.0141
110	17.4	do		24	24	48	155		24	9.5	2,600	3,160	5	2,940	5	2,720	93	10	−.0010
111	17.4	do		24	48	24	155		24	9.7	2,510	3,130	5	2,630	5	2,450	93	10	−.0002
112	17.4	do		24				[2] 27		9.9	1,650	2,630	5	3,540	5	1,790	51	10	+.0002
113	17.4	do		72					25	10.7	2,000	2,000	5	2,980	3	1,120	38		
114	17.4	do		24		48	100		25	10.1	2,270	2,790	5	3,040	3	1,040	34		
115	17.4	do		24		48	155		25	10.1	2,730	2,900	5	3,400	4	1,070	32	8	+.0416
116	17.4	do		24	48				25	10.3	2,220	2,470	5	2,500	5	2,430	97	10	+.0005
127	17.4	do		24		48	155		53	9.06	2,530	2,710	5	3,460	5	2,280	66	10	+.0012
128	17.4	do		24	24	24	155		53	9.08	2,350	2,480	5	2,830	5	3,180	112	10	+.0003
129	17.4	do		24		72	155		52	8.95	2,700	3,120	5	3,320	5	1,970	59	10	+.0009
130	17.4	do		24	24	48	155		52	9.07	2,750	2,720	5	2,830	5	3,080	109	10	+.0013
131	17.4	do		24	48	24	155		52	9.16	2,630	2,470	5	2,600	5	2,560	98	10	+.0008
132	14.4	Ciment electrique.[3]		24		48	155		25	8.10	2,250	2,910	5	2,760	5	2,790	101	10	−.0010
133	14.4	do		24	24	24	155		25	8.02	2,280	2,680	5	2,080	5	2,900	139	10	−.0013
134	14.4	do		24		72	155		24	7.98	2,470	2,850	5	2,440	5	2,840	116	10	−.0015
135	14.4	do		24	24	48	155		24	7.82	2,620	2,960	5	2,650	5	2,590	98	10	−.0013
136	14.4	do		24	48	24	155		24	7.77	2,550	2,930	5	2,630	5	2,580	98	10	−.0004
142	14.4	do		24				[4] 27		8.67	2,910	3,600	5	3,590	5	4,060	113	10	−.0004
143	14.4	do		72					25	9.34	4,220	4,430	5	4,220	5	4,310	102	10	+.0016
144	14.4	do		24		48	100		25	9.13	4,320	4,640	5	3,750	5	4,490	120	10	+.0022
145	14.4	do		24		48	155		25	8.11	2,200	2,760	5	2,780	5	2,620	94	10	−.0011
146	14.4	do		24	48				25	7.77	2,600	2,870	5	2,500	5	3,010	120	10	−.0008
248	10.4	Super cement [5]		24				[4] 20	35	9.70	1,430	2,770	5	3,730	5	3,310	89	10	−.0020
249	10.4	do		24				[4] 20	35	9.21	1,450	2,710	5	3,870	5	3,280	85	10	−.0022
250	10.4	do		24				[4] 20	35	9.48	1,410	2,720	5	3,650	5	3,240	89	10	−.0016
254	58.4	Ciment fondu [3]		24				[4] 20	35	9.23	5,030	4,540							
255	58.4	do		24				[4] 20	35	9.36	4,990	5,750							
[6] 276	27.4	Atlas lumnite [3]		24				[4] 20	35	6.33	6,210	7,680							
277	27.4	do		24				[4] 20	35	8.31	4,940	4,810							
278	27.4	do		24				[4] 20	35	8.28	5,190	4,840							
[7] 279	27.4	do		24				[4] 20	35	10.25	3,380	3,090							
[8] 280	27.4	do		24				[4] 20	35	11.75	2,160	1,920							
[8] 292	58.4	Ciment fondu [3]		24				[4] 20	35										
293	19.4	One-half Northwestern, one-half Universal.		24				[4] 20	35										
294	19.4	do	Sulphur impregnated.[9]	24				[4] 20	35										
295	19.4	do	Sulphur admixture.	24				[4] 20	35										

[1] For tests of cement see Table 5.

[2] Tap water.

[3] High alumina cement.

[4] Distilled water.

[5] Special cement.

[6] Standard Ottawa sand mortar cylinder except in mix, which is 1 : 2.

[7] Standard Ottawa sand mortar cylinder except in mix, which is 1 : 4.

[8] Standard Ottawa sand mortar cylinder except in mix, which is 1 · 5.

[9] At eighth week of curing the cylinders were placed, in melted sulphur for 15 minutes.

ratio of 97 per cent, as compared with the tank specimens, the minimum being 87 and the maximum 114 per cent. The fact that these cylinders after one year's exposure to the water of the lake average slightly weaker than the corresponding cylinders stored in the laboratory tank may be due to slight retardation of the natural curing processes, because of the lower temperatures of the lake water, or may be due to the influence on the curing of the sulphates in the lake, or to chemical action. In any event a strength ratio of 97 per cent is so near normal as to render further tests necessary before drawing definite conclusions.

3. Curing in water vapor at temperatures of 155° F. and 100° F. has been of no value. Ninety cylinders from 18 series in which a temperature of 155° F. was used had a strength ratio of but 38 per cent as compared with check cylinders from the laboratory tank, while 35 cylinders from 7 series in which a temperature of 100° F. was used in the curing had a strength ratio of 31 per cent. Actually the results are not even as favorable for a temperature of 155° F. as these figures would indicate, for when the strength ratios of 35 cylinders from 7 of the 155° F. series are compared with the same number of cylinders from the 100° F. series—the cylinders from the two groups being identical in all respects except that of the temperature of the curing vapor—the results are 21 and 31 per cent, respectively. This is a surprisingly poor showing for the cylinders cured at 155° F. It would naturally be expected that, since the cylinders cured at 212° F. showed up so well, those cured at 155° F. would be at least more resistant than those cured at 100° F. (see figs. 1, 2, 3, and 4). It is evident from this that there is a critical temperature point, somewhere above 155° F. in the curing of concrete made of standard Portland cement if the purpose be to develop resistance to sulphate waters.

4. Of the cylinders of series 92, 93, 112, 113, 172, 177, and 182 cured in the laboratory in the moist closet, followed either by further curing in water or by storing in air, or both, before exposure to the water of Medicine Lake, it is noted that the strength ratio of 50 such cylinders is 64 per cent when compared with cylinders from the same series stored in the laboratory tank. The temperatures under which these cylinders were cured ordinarily ranged around 70° F., and it appears that this temperature has been more effective than either 155° F. or 100° F. in retarding the action of sulphate waters (see fig. 4), although considerable deterioration is evident in all cylinders of these series after exposure to the lake water for one year.

5. In series 157 to 186, most of which contained admixtures, the condition of the specimens after one year in the lake indicates that under some conditions additions of blast-furnace slag, calcium chloride, Cal, and ironite have increased the resistance of the concrete while additions of volcanic ash and Alkagel "A" have been of little or no benefit. All these results are somewhat involved, and tests after longer periods of exposure will be necessary before conclusions will be of value. However, the curing conditions which produced the best result after one year's exposure of cylinders containing each of the foregoing admixtures are shown in Table 4, together with the results of tests for strength and change of length. No surface deterioration was evident in any of the cylinders of the series included in this table.

TABLE 4.—*Best curing conditions for specimens containing admixtures as indicated by tests after exposure for one year to Medicine Lake water*

Series No.	Admixture		Curing conditions after 24 hours in moist closet			Strength ratio, lake to tank specimens	Length change
	Amount	Material	Time in water	Time in water vapor	Time in air		
	Per cent		*Days*	*Days*	*Days*	*Per cent*	*Inch*
159	40	Blast-furnace slag	20		35	98	−0.0011
164	4	Calcium chloride	20		35	93	−.0003
169	4	Cal	20		35	105	−.0006
173	20	Ironite	20		35	102	−.0004
181	20	Volcanic ash		2	35	84	−.0001
186	3	Alkagel "A"		2	35	93	−.0006

SPECIMENS INSTALLED IN WOODEN CRATES

The specimens were installed in the lake in copper-nailed wooden crates, 50 cylinders to a crate, as shown in the illustrations. The crates were marked by copper tags and otherwise permanently identified by burned-in numbers, and each specimen had a positional identity within the crates. The crates were installed in the lake in about 5 feet of water; consequently the specimens were not subjected to any weathering action due to freezing and thawing.

In general, the cylinders were crated and shipped by express to Medicine Lake when 8 weeks old and were placed in the lake within three days after leaving the laboratory. Those cylinders which were not cured 8 weeks were placed in earthenware jars containing 10 liters (2.6 gallons) of Medicine Lake water, 20 cylinders to a jar, and at 8 weeks were crated and shipped to Medicine Lake. On the same day the cylinders were placed in Medicine Lake water other cylinders of the same series were placed in tap water in the laboratory tank.

In all cases the mix used in the cylinders was proportioned by volume with the weight of cement assumed at 100 pounds per cubic foot. Where an admixture was used the amount of cement and aggregate remained the same and the material added was measured as a percentage by weight of the cement used. Each batch was mixed by hand at least 1½ minutes dry and 2 minutes after adding the water. The materials, after mixing, were tamped in three 3-gang brass molds in four layers, each layer being tamped 20 times. For tamping the laboratory standard cylinders a pointed rod was used which was three-eighths inch in diameter by 15 inches long; for the standard Ottawa sand cylinders a flat-ended ⅝-inch rod was used which weighed 337 grams. Round-head brass screws 1 inch long were set in neat cement on the ends of three cylinders, and measurements between these screwheads served as the basis for the record of change of length. The other six cylinders of each batch were left with blank ends and were used in the various compression tests.

TESTS OF CEMENT AND AGGREGATES

The aggregate, mix, and consistency of all laboratory standard cylinders were identical with the few exceptions noted in Table 2, the variables being curing, cements, and admixtures. The cement used, unless otherwise stated in Table 2, was a mixture of equal parts of Northwestern and Universal Portland cement. For physical tests of all cements used see Table 5.

Table 5.—*Summary of tests of cements used in experimental cylinders placed in Medicine Lake*

Laboratory No.	Lot No.	Brand of cement	Time of set								Steam test	Fineness	Water required for normal consistency	Specific gravity	Tensile strength of standard briquettes		
			Vicat				Gilmore										
			Initial		Final		Initial		Final						3 days	7 days	28 days
			Hours	*Minutes*	*Hours*	*Minutes*	*Hours*	*Minutes*	*Hours*	*Minutes*		*Per cent*	*Per cent*		*Pounds*	*Pounds*	*Pounds*
8.4	8	One-half Northwestern and one-half Universal	3	10	6	20	4	10	7	20	O. K.	21.6	24.0	3.11	149	230	342
10.4	------	Super cement	4	40	18	30	5	30	18	30+	O. K.	7.7	24.25	--------	122	184	324
11.4	11	One-half Northwestern and one-half Universal	2	45	5	40	3	25	7	0	O. K.	17.9	23.0	3.13	150 131	235 186	357 316
14.4	------	Ciment electrique [1]	3	50	6	35	4	35	6	45	O. K.	16.8	26.0	3.22	278	316	--------
17.4	12	One-half Northwestern and one-half Universal	2	15	6	0	3	40	6	15	O. K.	20.0	25.0	3.13	130	217	317
18.4	13	do	3	15	6	50	5	30	7	20	O. K.	22.0	25.0	3.13	126 168	186 243	307 382
25.4	------	Atlas lumnite [1]	9	0	11	0	9	0	11	0	O. K.	7.8	22.5	--------	466	458	472
27.4	------	do	5	10	7	10	5	30	7	25	O. K.	14.0	22.5	--------	465	452	398
28.4	------	Super cement	----	----	----	----	----	----	----	----	----	----	24.25	--------	109	176	281
33.4	------	Lehigh	3	30	5	25	3	10	5	25	O. K.	17.9	22.5	3.13	209	259	387
34.4	------	Hawkeye	5	20	7	55	6	10	8	40	O. K.	9.1	28.5	3.14	224	285	403
58.4	------	Ciment fondu [1]	5	0	7	15	5	30	7	55	O. K.	6.6	22.5	3.11	439	424	376
61.4	15	One-half Northwestern and one-half Universal (mixture of Nos. 49.4, 50.4, 43.4, and 44.4)	4	40	7	20	5	5	7	30	O. K.	14.5	23.5	3.12	151	218	336
62.4	16	Universal (mixture of Nos. 43.4 and 44.4)	4	50	7	5	6	25	7	25	O. K.	17.0	24.0	3.14	172	231	326
63.4	17	Northwestern (mixture of Nos. 49.4 and 50.4)	4	50	7	30	6	15	8	5	O. K.	18.2	23.5	3.18	134	218	335

[1] High-alumina cement.

The aggregate, obtained from the J. L. Shiely Co. gravel pit at St. Paul, Minn., passed all standard physical tests, including a test for shale, which showed less than 0.4 per cent by volume. The various sizes were separated by screening and recombined for each batch, as shown in Table 6. The combination used in series Nos. 77 to 106 differed slightly from the normal but the fineness modulus was 4.67.

Table 6.—*Screen analysis and quantities of aggregate for each 9-cylinder batch of laboratory standard cylinders*

(Unit room-dry weight of aggregate, 124 pounds per cubic foot)

Screens	Percentage	Coarser than—	Percentage	Amount required
				Grams
Retained on ⅜ inch	0.0	⅜ inch	0.0	----------
Passing ⅜ inch, retained on No. 4	38.2	No. 4	38.2	1,350
Passing No. 4, retained on No. 8	25.8	No. 8	64.0	910
Passing No. 8, retained on No. 16	13.3	No. 16	77.3	470
Passing No. 16, retained on No. 30	13.3	No. 30	90.6	470
Passing No. 30, retained on No. 50	6.8	No. 50	97.4	240
Passing No. 50, retained on No. 100	2.4	No. 100	99.8	[1] 91
Passing No. 100, retained on pan	.2	----------	----------	----------
	100.0	----------	467.3	[2] 3,531

[1] Passing No. 50 and retained on pan.

[2] This quantity is for a 1 : 3 mix; for a 1 : 4 or 1 : 5 mix the quantities were increased 10 per cent for each size of aggregate, making a total of 3,884 grams.

Fineness modulus=4.67.

The standard Ottawa sand cylinders differed in no respect other than the aggregate from the standard concrete cylinders. As in the latter, the standard mix was 1 : 3 and the relative consistency 1, while the variables, as in the other group, were curing, cements, and admixtures. The cement used, unless otherwise stated in Table 3, was a mixture of equal parts of Northwestern and Universal Portland cement.

Compression tests were made in a 50,000-pound Reihle machine, using a bottom spherical bearing block. The cylinders tested at 7 and 28 days were cured in the same manner as the cylinders to be placed in Medicine Lake except those which were intended to be cured 24 hours in the moist closet and 20 days in water. These were kept in water until tested.

The absorption test results are based on the A. S. T. M. standard absorption test for drain tile, which stipulates drying the specimen to complete dryness in a temperature of not less than 230° F., followed by boiling for five hours.

DESCRIPTION OF ADMIXTURES USED

Series 157 to 161, blast furnace slag.—The slag used in these series was obtained from the Universal Portland cement plant at Duluth, Minn. Before using, it was ground in a coffee-mill grinder in the State Bureau of Mines at the University of Minnesota until it approached the fineness of cement. The mill used was not heavy enough to crush the iron pebbles and these were rejected. The material, after grinding, was screened on sieves Nos. 100 and 200 and recombined in the proportion of 1 part passing the No. 100 and retained on the No. 200 sieve and 5 parts passing the No. 200 sieve. It was mixed dry with the cement and aggregate.

Series 162 to 166, calcium chloride.—The calcium chloride used in these series was a chemically pure product, dissolved in the mixing water.

Series 167 to 171, Cal.—Cal, used in these series, is the trade name for a powdered manufactured product. Quoting from United States Bureau of Standards, Technologic Paper No. 174, "Cal is a material obtained by pulverizing the dried or undried product resulting from a mixture of either quicklime or hydrated lime, calcium chloride and water."

Series 172 to 176, ironite.—This is the trade name for a powdered waterproofing product. The recommended method of use is by brush coat or coats, applied to the surface of the concrete after the removal of the forms. In series 173, 175, and 176 it was mixed dry with the cement and aggregate. Such use of this material as an admixture, so far as is known, is not advocated by the manufacturers.

(Continued on page 183)

CLAY SOILS IN RELATION TO ROAD SUBGRADES

By HUGH H. BENNETT, Soil Scientist, United States Bureau of Soils

According to tests which have been recently made,[1] unfavorable road subgrade conditions are frequently due to the presence of an excessive amount of clay soil of a plastic nature. Ordinarily the quantity of the clay present in the soil determines the quality of the subgrade, but there are a number of exceptions to this rule, particularly in the soils of tropical regions and certain soils of the more southerly latitudes in the United States. In this connection some observations relating to fundamental differences in the character of clays, with emphasis on their effect upon stability or serviceability as subgrade material,[2] may be of interest to highway engineers.

THE CHARACTER OF CLAY SOILS IN THE UNITED STATES

Clay,[3] as commonly known, is a very fine-grained and dense soil which absorbs water slowly and holds it tenaciously because of the lag of surface tension and other forces which impede the movement of moisture through the minute pore spaces between the grains. Well-known properties of clay include pronounced stickiness and plasticity when wet, and hardness, often accompanied by shrinking and cracking when dry. These conceptions will usually fit the conditions in the humid sections of the United States, although there are some variations in these characteristics in different localities, corresponding with the differences between the soil types.

It might be supposed from the accepted definition that those clays having the higher percentages of sand would be also the least impervious to water. This, however, is not invariably true. Permeability is often due more to the degree of the weathering of the soil and the consequent character of the colloids [4] rather than to the size and textural grading of the constituents. In some highly weathered clays, such as exist in the more southerly latitudes, the flocculent character of the extremely fine particles provides a more open and better-drained soil, even in the case of extraordinarily fine-grained clays, than is found in more northerly regions, even in sandy clays, where the colloids are not of the kind that cling together to form soil aggregates.

OUTSTANDING ILLUSTRATIONS OF ADVANCED WEATHERING

The most outstanding illustrations of the effect of advanced weathering upon soil character are to be found in the humid tropics, where, under a constantly warm temperature and heavy rainfall, soil weathering over large areas has proceeded to a stage considerably in advance of that prevailing in the North Temperate Zone. There are soils in Central America with 90 per cent of the particles as small as the finest clay that are as friable and open-natured as the mellowest loam. On the contrary, clays approximating this fineness in the United States may be counted upon to be exceedingly impervious to water, subject to swelling and shrinking with variations in moisture content, and of a poor quality for highway subgrades.

Humid tropical clay—that is, the older clay, which has existed in place for a long time—has the permeability previously referred to. This condition results chiefly from the weathering of the colloids until they are characterized by a marked tendency of the particles to group into clusters, from which the individual grains strongly resist detachment by water. This aggregating of the minute particles gives rise to an open, granular structure, while the refusal of the grains to break away and float off in water prevents the soil pore space from clogging through a process of downward transposition of the particles by percolating water (eluviation). Nearly all the rainfall passes downward through the soil, and the little which runs off usually shows only a slight turbidity. In Cuba the famous red soil of Matanzas and other provinces absorbs the rain to such an extent that over broad areas no streams whatever have developed.

The soils of the humid tropics show very little tendency to swell and shrink with variations in moisture content, they strongly resist erosion, and are only slightly sticky when wet. In extreme cases, as in eastern Costa Rica, even plowing in the rain results in a fine tilth.[5] Material of this kind is usually a product of the climate and the duration of the weathering. Friability and resistance to erosion are much more nearly normal properties of the fine soils of a warm, wet climate than of soils in a cool, wet climate. The plastic nonfriable clays of the wet tropics, and frequently also of the Temperate Zone, are the young clays, such as poorly drained alluvial clay and freshly decomposed shale material.

DELINEATION OF THE REPRESENTATIVE MELLOW-CLAY AREAS OF THE UNITED STATES

There are in the United States no extensive areas of mellow clays precisely like the severely weathered types of the tropics, but there are large areas, notably in the Piedmont Plateau from New Jersey to central Alabama, in which the clay soils have, in some degree, similar properties. Studies have shown that the colloids in the red granitic soils of the Piedmont (Cecil soils) have some of the characteristics or an approximation of the characteristics of the tropical colloids. The Cecil soils, including chiefly the clay, clay-loam, and sandy-loam types, have everywhere a brittle, red-clay subsoil, usually containing some particles of quartz derived from the parent rocks. In places quartz veins cut through the clay, these representing the most resistant part of the original country rock. Often the underlying granite or gneiss in its upper weathered part consists of "rotten rock," i. e., friable material which *in*

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[1] Practical Field Tests for Subgrade Soils. PUBLIC ROADS, vol. 5, No. 6, August, 1924.
Field Methods Used in Subgrade Soils. PUBLIC ROADS, vol. 6, No. 5, July, 1925.

[2] This article will discuss the direct effect of the grading and size of the soil grains and not the indirect effect of hardpan, compact layers, etc., upon the behavior of the overlying material under varying conditions of moisture. Nor will sandy clays be discussed, since these are usually permeable and stable.

[3] Clay soils, according to the textural definition, have the following proportions of ingredients: 30 to 100 per cent clay particles (diameter range, 0.005 mm. and less); less than 55 per cent sand (1 to 0.05 mm.); less than 55 per cent silt (0.05 to 0.005 mm.); and more than 50 per cent silt and clay.

[4] Exceedingly fine particles of the soil, sometimes referred to as ultraclay.

[5] Soils of this group are related to that kind of laterite which has a high content of the hydroxides of iron and alumina, and a relatively low content of silica and the bases (lime, magnesium, potash, and soda).

situ has preserved the skeleton structure of the original, unaltered rock.[6] Probably the most valuable properties of these soils, from the standpoint of the highway engineer, are the slight tendency to swell and shrink on wetting and drying, and a sufficient permeability to prevent water-logging, provided ordinary drainage outlets are maintained. The clay hardens on drying, but seldom cracks, even in cuts exposed to the air.

Essentially the same properties characterize the Porters soils of the Blue Ridge Mountain region. These are reddish brown at the surface, with clay subsoil much like the subsoil of the Cecil. They also are derived from granite and gneiss, show little tendency to shrink or swell, and admit of fairly free passage of water. On soil of this nature subgrade difficulties are likely to be of negligible importance. Clays of uniform red, brown, and deep-yellow colors are better oxidized than those of whitish, grayish, bluish, and pale-yellow colors, or mottlings of these colors, and they almost invariably are better drained, shrink less on drying, swell less on wetting, and are much firmer (less miry) in the presence of excessive moisture. The colloidal clay of the slightly oxidized light-colored and mottled soils has more of the swelling and shrinking properties, and the soils consequently are characterized by larger volumetric changes under varying moisture conditions. In dry weather an experienced soil man can readily identify clays such as the Susquehanna, Montrose, Iredell, Lufkin, Coxville, and Eldon, wherever exposed long enough to dry out, by the peculiar cracking ("checking") occasioned by shrinking.

REPRESENTATIVE HEAVY CLAY AREAS IN THE SOUTHERN STATES

Water passes slowly through the light-colored and mottled plastic clays; they become saturated with penetrating moisture, and eventually soft and unstable. These properties have given rise to such local names as "pipe clay," "beeswax," "soggy soil" (or "sobby soil"), "spouty clay," and so on. Even some of the mottled clays containing considerable sand, such as the Leaf (a stream-bench soil of the Coastal Plain region east of the Coastal Prairies), and the Elkton of the Chesapeake Bay region, behave unfavorably between the wet and dry ends.

The clay of the plastic, impervious Susquehanna soils is poorly oxidized, as shown by its mottled red, yellow, and bluish color. Frequently the upper part is red, with mottled clay beneath a thinly oxidized zone, the red color decreasing with the depth. These are very extensive soils in the Coastal Plain region, east of central Texas. Especially are they of abundant occurrence in west-central Georgia, southern Alabama, throughout the uplands of Mississippi, southern Arkansas, and east Texas. They are also scatteringly present as far north as the Susquehanna River locality of Maryland. The clay type and the clay subsoil of all types constitute unfavorable soil material for any purpose. When saturated it is slippery, plastic, and extremely miry; on drying it hardens, shrinks, and cracks. The Montrose clay occurring in association with the Susquehanna, on flat areas, has the properties of the latter, with some accentuation of the bad features, due to retarded oxidation caused by poor surface drainage. No red color is present in the subsoil, which consists of yellow and bluish-gray clay of extreme density and imperviousness. Quite similar features are exhibited by the gray Lufkin clay of the flatwoods regions of Texas, Arkansas, Mississippi, eastern North Carolina, and other parts of the Coastal Plain, and also by the clay subsoil of the Coxville types occurring through the Atlantic flatwoods from southeastern Georgia northward.

The Iredell soils, locally referred to as "pipe clay," "bull tallow," and "black jack" land, are the most extensive bad road soils of the Piedmont region. They have been formed from the decay of basic igneous rocks, chiefly diorite. Their subsoils consist of yellow or greenish-yellow, very sticky, plastic clay, which is impervious to water and swells or contracts as the water content rises or falls. The clay on drying always cracks badly.

In those parts of the Piedmont underlain by sedimentary rocks of Triassic age certain claystone horizons give rise to an unfavorable clay, White Store clay, that practically duplicates the properties of the Susquehanna clay. In the prairie regions of west-central Missouri the subsoil of the dark-colored Eldon series also essentially repeats the characteristics of the Susquehanna, including the unfavorable features relating to subgrades.

CONCLUSION

Space does not permit discussion of all the numerous clay types, most of which probably will be found unsatisfactory as subgrades for highways. However, it is believed that an application of the principles involved in the varying properties outlined above to the clay soils should roughly classify most of them as to their suitability or unfitness for subgrade material, at least for the humid regions of the United States. With these properties in mind, certain clays probably can be eliminated at once from the list of suitable materials, others can immediately be set down as favorable, while those of doubtful classes can be subjected to the necessary physical tests.

It should be observed in connection with texture that certain materials occurring in both the humid and arid regions, much more generally in the latter, will show in their physical analysis a composition as fine as the heaviest clay and yet have markedly different properties from normal clays. These are the chalky materials composed largely of lime carbonate or of lime sulphate or of both, and known locally by such names as "caliche," "plains marl," "gyp-beds," and "rotten limestone."[7]

All of these are low in content of those colloids that impart plasticity and water-logging properties common to the plastic clays. With plentiful moisture, the tendency is to remain firm rather than to soften. They are not true clays, and under normal conditions with respect to artificial removal of excess rain water they probably will function satisfactorily as subgrade material for roads.

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[6] This material has been used with good results for surfacing roads.

[7] The "rotten limestone" variety of chalk is largely found as the basal formation beneath the Houston soils of the prairie region of Alabama and Mississippi (Selma chalk) and the Black Waxy Belt of central Texas (Austin chalk). The other classes of materials outcrop or lie near the surface over vast stretches of the Great Plains and arid regions. A subsoil accumulation of lime of the caliche order really marks the boundary between the prairies and plains.

HIGHWAY RESEARCH BOARD ACTIVITIES

EXPLAINING the service rendered by the Highway Research Board of the National Research Council in a recent radio address under the auspices of the National Research Council and Science Service, Prof. S. S. Steinberg, of the University of Maryland, remarked that whereas a few years ago a road contract 2 miles long was considered a large undertaking, contracts are now let in 10 and 20 mile sections. In one day recently, according to Professor Steinberg, Pennsylvania let road contracts totaling $12,000,000. The previous record was that of Illinois, with a letting of $9,000,000 in one day. Such stupendous programs, it has been truly said, will make the construction of highways the most active American industry during the current year.

Continuing, Professor Steinberg said that—

> Fortunately, highway engineers realize the necessity of having this great development of our highway system proceed on sound principles of engineering and economics. This is evident from the fact that the Federal Government, the State highway departments, and many universities are engaged in an extensive program of research into the problems affecting highway finance, construction, and maintenance. A recent census shows that there are almost 500 highway research projects under way throughout the country. It is the function of the Highway Research Board of the National Research Council to coordinate these researches, spread as they are over such a wide territory; to prevent duplication of effort, by putting the workers into touch with one another; and finally, to make known to each State the findings which may be immediately applied in practice; thus resulting not only in the better construction and maintenance of highways but also in a great saving in the taxpayer's dollar.
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> The Highway Research Board thus occupies a unique position as a service organization to highway engineers and to the public, and it enjoys the complete confidence of all agencies throughout the country engaged or interested in highway development. As indicative of the personnel of the board and interests represented, we might cite that Dean A. N. Johnson, of the University of Maryland, is chairman; W. H. Connell, of the Pennsylvania State Highway Department, is vice chairman; and C. M. Upham, State highway engineer of North Carolina, is director. The executive committee also includes Prof. T. R. Agg, of Iowa State College; A. J. Brosseau, president of Mack Trucks (Inc.); Dr. H. C. Dickinson, of the United States Bureau of Standards; Thomas H. MacDonald, chief of the United States Bureau of Public Roads; and W. Spraragen, of the National Research Council.

CONCRETE REINFORCEMENT STUDY NEARING COMPLETION

> Where research of a general character needs to be undertaken, which a single organization would not be justified in conducting, the Highway Research Board has been assigning an expert to carry on such investigation. The results are then made available to all interested agencies. As an example, there is now nearing completion a nation-wide survey of the economic value of steel reinforcement in concrete roads. In some States it is the practice to incorporate in the concrete steel rods or mesh, using half a pound or more to every square foot of surface. In other States no reinforcement is used. As this item alone may amount to many thousands of dollars per mile of road, it is necessary for the highway engineer to know whether the use of reinforcement is justified, and if so, the proper amount needed for the greatest economy.
>
> Another investigation, just commenced under the board's auspices, deals with the development of earth roads. Although we may now travel with speed and comfort from one end of the country to the other, we must not forget that 85 per cent of all our rural highways are still in a state of nature, and many of them impassable for several months of each year. It may help us to realize what a great economic loss this means when we consider that one-third of all the automobiles in use are owned by farmers most of whom are compelled to use these earth roads. This problem is an especially significant one in the Western States where there is a great mileage of highways to improve and maintain and the funds are so limited that any extensive program of surfacing with the more costly types of pavements is out of the question.
>
> Many attempts have been made to solve this problem. For instance, in North Carolina, where there is a sea of sand stretching for hundreds of miles along the coast, the State Highway Commission has developed, as a result of research, a sand-asphalt surface made up of a mixture of 93 per cent local sand and 7 per cent asphalt. This renders very satisfactory service to that locality. In South Carolina, Illinois, and California, tars and oils are being used, either by mixing with the earth or as surface applications. In the Southern States either the top soil of the fields or a mixture of sand and clay is used. In Iowa, Missouri, and South Dakota experiments are being conducted in which the natural soil is mixed with hydrated lime, or in some cases with Portland cement, in an attempt to stabilize the natural soil.
>
> Realizing that this is one of the most important problems confronting highway engineers, the Highway Research Board is attempting to coordinate completed and current research on this subject with the hope of developing a low-cost road surface that will be suitable for light traffic.

Professor Steinberg, who has been designated as the acting secretary of this new investigation, has been head of the department and professor of civil engineering at the University of Maryland since 1918. He was previously connected with the New York, Delaware, and South Carolina highway departments, and served also on subway construction in New York and railroad construction in Central America. During several summers he has been employed by the Bureau of Public Roads on details at the University of Maryland in connection with research conducted there by Dean A. N. Johnson. During the summer of 1924 he was assistant director of the Highway Research Board.

Another investigation which has been begun under the auspices of the board is that on culvert pipe. The objects of this study are to correlate the various tests and experiments on pipe loads and stresses, to set up a standard with which to compare the life of different kinds and classes of culvert pipe, and to determine the most probable conditions of service for which standards should be designed.

CRUM APPOINTED TO HEAD CULVERT PIPE INVESTIGATION

R. W. Crum, engineer of materials and tests, Iowa State Highway Commission, has been designated as chairman of the culvert-pipe investigation. Mr. Crum was for 12 years associate professor of civil engineering at the Iowa State College, during which time he was engaged on research work for the State engineering experiment station. He has held his present position with the State highway commission since 1919. He is the author of a number of important research papers, and is an active member of several research committees of the American Society of Civil Engineers and the American Society for Testing Materials.

In view of the constantly enlarging program of highway development in Latin America, the board is extending a cordial invitation to the Latin American universities to designate honorary representatives, who shall serve as liaison officers between the universities and the board. Similar contacts now exist with the universities in the United States.

These invitations have been presented personally by Dean A. N. Johnson, who is now in South America as one of the United States delegates to the Pan American Congress of Highways held in Buenos Aires in October.

THE PREPARATION OF CONCRETE CORES FOR COMPRESSION TESTS

Reported by C. E. PROUDLEY, Assistant Engineer of Tests

THE division of tests of the Bureau of Public Roads has received a number of requests for information relative to the best method of preparing concrete cores for compression tests. The following information may be of assistance to those who have already taken up this line of investigation, as well as those who anticipate doing so in the future.

It has been found that capping the cores with neat cement is the most satisfactory method, whether the ends of the core are very irregular or not. Plaster of paris has been used with success where the specimens had especially smooth end surfaces, but in general this practice is not advised. It has been found from comparative tests that cores capped with plaster of paris may show a reduction in strength as great as 50 per cent below those similar in character capped with Portland cement.

The method adopted in the laboratory of the Bureau of Public Roads is to remove any very sharp or abnormal projections from the end of the core by means of a carborundum saw or the skillful use of a hammer, after which the surface is cleaned with hydrochloric acid, followed by rinsing with a jet of water. Unless this cleansing process is carefully done the cap will not adhere to the core. No attempt should be made to prepare specimens with heights equal to or exactly twice their diameters. It has been found that more accurate results are obtained when the original core is capped and tested without any attempt to build up its height with the capping material. The necessary correction for height may easily be made by applying the correction factor given in A. S. T. M. Tentative Standard Method of Securing Specimens of Hardened Concrete from a Structure, serial designation C45–21T.

The surface to be capped is thoroughly brushed with a soft paste of neat cement in order to insure a good bond for the cap. The specimen should be kept wet at all times, and a minimum period of time should elapse between the various operations in order to prevent premature drying of the unhydrated cement in the cap. Several methods of applying the cap may be followed, depending upon the irregularities of the surface. If the end of the core is comparatively smooth neat cement paste of approximately normal consistency is heaped upon the specimen and the latter then inverted onto a piece of plate glass which has been oiled and is free from scratches. New plate glass is recommended, as any scratches or roughness may cause the cement to stick to the glass, thereby removing the cap from the core. The excess cement is squeezed out by pressing the core firmly down against the plate, and a trowel is used to point up the cement around the edges of the core. Care should be taken that the axis of the specimen is perpendicular to the glass surface.

Another method which is used in the case of specimens having irregular ends consists of wrapping a piece of thin sheet metal, which has been oiled, around the core so as to form a mold. This may be held in place with several wire bands. The highest point of the end of the core should be flush with the end of this mold, so that when the cement paste is applied there will be neither an excessive thickness of cap nor any projection from the core. The form thus prepared is filled with cement paste, after which a piece of oiled plate glass is placed upon it so as to form a plane bearing surface.

The selection of oil in these operations is important, as some oil seems to lack those qualities which prevent adhesion of the cement to the glass or metal surfaces. Several brands should be tried for the purpose of selecting that which gives the best results. Mobiloil B and tractor crank-case oil have been used in the bureau's laboratory with satisfaction.

Lumnite cement, either as a stiff paste or in a 1: 1 mortar mixture, should give satisfactory results if sufficient care is taken to supply all the water required for its hydration. When Lumnite cement is used the freshly capped specimens should be kept in a moist closet or thoroughly surrounded by saturated burlap bags for at least 12 hours. If the initial curing is done properly, cores capped with Lumnite cement may be tested 24 hours after capping. Portland cement paste gives satisfactory results, especially if a 4 per cent solution of calcium chloride is used as the mixing water. This accelerates the hardening of the cement sufficiently to allow a compression test to be made after three days of curing.

The chief requisites of a material suitable for capping specimens are that its strength at the time of the test shall be at least equal to that of the specimen and that the modulus of elasticity shall be as nearly that of the concrete under test as it is feasible to obtain.

(Continued from page 179)

Series 177 to 181, volcanic ash.—The ash is a siliceous material from Sheridan County, Nebr. It was mixed in the batch dry with the cement. Insoluble in cold water and free from organic impurities, it has the following screen analysis:

Screen sizes	Per cent
Passing No. 30, retained on No. 50	0.0
Passing No. 50, retained on No. 100	0.4
Passing No. 100, retained on No. 200	8.4
Passing No. 200	91.2
	100.0

Series 182 to 186, Alkagel "A".—This is the trade name for a colloidal product manufactured at Duluth, Minn. It was dissolved in the mixing water and added to the batch.

Series 294 to 295, sulphur.—Powdered commercial sulphur was used in series 295 and the same material was used in the impregnated series 294.

COMMITTEE ON TESTS AND INVESTIGATIONS MEETS

THE subcommittee on tests and investigations of the committee on standards of the American Association of State Highway Officials met in Washington at the Bureau of Public Roads on September 29 and 30.

A part of the time of the committee was taken up with discussion of revision of United States Department of Agriculture Bulletin No. 1216, "Tentative Standard Methods of Sampling and Testing Highway Materials." Several changes relating to methods of procedure were recommended. The new standard screen scale for mechanical analysis of fine aggregate for concrete recently adopted by the American Society for Testing Materials was also adopted by the subcommittee. The former requirement was for screens with square openings, whereas round openings are now optional for the ¼-inch size and larger. The new screen scale permits of a greater variety of sieves than the old scale.

The committee received and discussed several research reports, such as Investigations of the Relation Between the Transverse and Compressive Strength of Concrete, The Value of Stone Screenings in Place of Sand as Fine Aggregate in Concrete, The Use of Rail Steel for Reinforcing in Concrete, Methods of Determining the Proportions of Hardened Concrete, Methods for the Determination of the Asphaltic Content of Road Oils, and Methods for Determining Toughness of Bituminous Mixtures. Specifications for concrete culvert pipe and for paints were also discussed.

The following members were in attendance:

H. A. Hall, Florida.
H. F. Clemmer, Illinois.
R. W. Crum, Iowa.
Mark Morris, Iowa.
F. C. Lang, Minnesota.
W. F. Purrington, New Hampshire.
J. G. Bragg, New Jersey.
F. H. Bauman, New Jersey.
W. L. Blaum, New York.
J. E. Myers, New York.
A. S. Rea, Ohio.
R. I. Giles, North Carolina.
O. H. Hansard, Tennessee.
J. V. Keily, Rhode Island.
H. S. Mattimore, Pennsylvania (chairman).
M. H. Ulman, Pennsylvania.
Shreve Clark, Virginia.
R. B. Dayton, West Virginia.
J. R. Boyd, Bureau of Public Roads.
I. B. Mullis, Bureau of Public Roads.
A. C. Rose, Bureau of Public Roads.
L. G. Carmick, Bureau of Public Roads.
H. M. Milburn, Bureau of Public Roads.
F. H. Jackson, Bureau of Public Roads (secretary).
F. H. Janda, National Research Council.

(Continued from page 168)

BOTTOM EDGE TENSION DOUBLE TOP CORNER TENSION

The data indicate that with a load passing along a path 9 inches from the edge of the pavement the tension produced in the top of the slab by a load at the corner is always less than the tension in the bottom of the slab directly under the wheel. The relative magnitude of these two tensions was compared by expressing the deformation recorded along the diagonal in the top of the slab as a percentage of the average maximum deformation recorded by four gauges in the bottom edge of the slab with the following result:

Wheel load	Ratio of stress along diagonal to stress along edge
	Per cent
4-wheel truck:	
4,000 pounds	42
5,000 pounds	40
6,000 pounds	45
7,000 pounds	41
8,000 pounds	57
6-wheel truck:	
Average all loads ... pounds	48

The gauge in the diagonal was located 48 inches from the corner, or about 35 inches from the wheel, when the maximum stress was produced. Shifting the position of this gauge along the diagonal might change the recorded stress, although data from previous tests were used in locating it. These percentage values are merely indications, and are included only to give an idea of the approximate relation between these stresses in a uniform slab where there is little or no support across the transverse joint.

Summing up, the following may be stated as the indications of this investigation:

1. The deflection of a concrete pavement is directly proportional to the load applied (within the limits of this investigation).
2. A load passing along the test pavement (uniform thickness, 6 inches) 9 inches from the edge of the pavement produced approximately twice the fiber deformation in the edge of the pavement that was caused by the same load passing along a path 21 inches from the edge.
3. The tension produced in the top of a pavement due to counterflexure between the wheels of a six-wheel vehicle is less than the tension produced in the bottom of the pavement directly under the wheel regardless of the axle spacing.
4. In the case of six-wheel vehicles the maximum tension produced in the pavement seems to be a function of the wheel load and not of the axle spacing, at least between the limits of 3 feet and 10 feet.
5. Within the deformation limits obtained in this investigation (maximum unit fiber deformation of about 0.0001 inch per inch) the fiber deformation in the pavement is directly proportional to the load.
6. In a pavement slab of uniform thickness the maximum deformation occurs along the edge of the slab for both four-wheel and six-wheel vehicles.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets or to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916. 10c.
*660. Highway Cost Keeping. 10c.
670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691. Typical Specifications for Bituminous Road Materials. 10c.
*704. Typical Specifications for Nonbituminous Road Materials. 5c.
*724. Drainage Methods and Foundations for County Roads. 20c.
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
No. 1259. Standard Specifications for Steel Highway Bridges adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULAR

No. 94. TNT as a Blasting Explosive.

FARMERS' BULLETINS

No. *338. Macadam Roads. 5c.
*505. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *727. Design of Public Roads. 5c.
*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

SEPTEMBER 30, 1925

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863.197.86	611.8	$ 2,596,734.31	$ 1,227,424.29	134.4	$ 13.119,144.41	$ 6,363,120.19	694.2	$ 378,592.64	$ 175.050.00	11.6	$ 2.179.863.66	Alabama
Arizona	9,580,133.43	6.016.119.94	613.8	237.926.80	145.397.06	7.0	1,969.799.31	1,249,914.42	157.3	276,880.71	180.420.97	35.5	1.969.488.61	Arizona
Arkansas	13,310,190,08	5.380.181.73	1048.9	1,680,021.97	749.315.78	60.1	6.722.805.84	3,147,432.39	415.0	1,076,958.63	506.303.04	85.8	554,664.06	Arkansas
California	22,346,175.99	10.719.249.61	894.8	1.026,130.48	509,752.08	24.4	10,429.810.75	6,129,483.09	270.5	1,650,336.56	732,415.86	89.1	2,497.208.36	California
Colorado	11,876,703.94	6.067,814.34	651.2	663,724.78	301.210.67	20.9	4,592.631.05	2.283,554.88	222.6	196,031.87	109,947.34	13.6	2.182,900.77	Colorado
Connecticut	4,558,639.29	1,819,368.66	101.6	575,452.78	171,947.14	9.5	1,909,405.24	609,235.48	29.4	38,649.99	19,324.00	1.5	1,240,377.72	Connecticut
Delaware	4,281,559.81	1.496.190.65	107.1	383.163.86	180,683.85	12.1	1.066.711.56	420.745.86	22.9				10,912.66	Delaware
Florida	2,959,273.72	1.405.487.97	96.3				8.970.318.13	4.370.709.60	251.8	593.589.09	204,181.77	11.8	1.207.389.66	Florida
Georgia	20,156,002.37	9,406,366.46	1478.3	1,533,977.67	752,849.49	108.9	11,197,627.34	5,428,240.05	693.9	1,386,866.86	435,218.24	91.0	428,089.76	Georgia
Idaho	9,394,676.80	4.815.332.26	600.1	176.117.58	101,367.48	21.0	2.711.794.26	1.707,146.35	176.5	680.788.60	411.380.96	37.3	687.810.95	Idaho
Illinois	40,010,481.10	18,640,076.28	1236.2	688.967.89	340.741.22	25.0	8,598.101.74	4.165.939.64	289.9				3,509,824.86	Illinois
Indiana	13,639,172.65	6,562,455.68	422.1	1,132,976.57	544,542.84	41.2	17,042,952.77	7,969,148.90	471.2	112,160.81	56,080.40	4.2	1,136,237.18	Indiana
Iowa	27,272,285.21	11,107,492.99	1996.9	102.280.50	39,495.95	16.3	7.671.624.74	3.441,026.98	463.7	1.502,174.74	696.327.65	122.9	2.140,750.43	Iowa
Kansas	26,399,695.77	9.755,273.32	831.4	2.492,110.88	1.067,579.28	115.1	10,837,265.30	4.627,627.02	530.9	1.532,784.09	465.023.99	103.6	1.476.741.39	Kansas
Kentucky	14,832,324.28	6,205,994.59	584.9	1,356,407.10	565,020.08	45.6	7,950,326.58	3,550,910.53	306.1	79,653.87	39,826.93	7.0	1,434,247.87	Kentucky
Louisiana	11,939,424.97	5,279,870.86	927.6	875.723.58	411.312.04	65.0	2,772.603.62	1,325,240.49	108.8	567,118.64	283,056.89	21.6	972.163.72	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	173,275.15	86,733.59	6.6	1.311,335.31	651.480.38	42.2	973,159.26	392,833.36	35.6	843.336.34	Maine
Maryland	8,132,506.90	3,849,383.16	294.4	271,012.37	128,725.85	9.7	2,919,045.57	1,293,531.65	131.3				18,792.36	Maryland
Massachusetts	14,047,656.22	5.467,661.28	300.6	505.080.98	165.015.60	11.0	5.894.811.24	1.566.221.98	84.9	523,237.49	173,035.13	8.1	1,647,737.01	Massachusetts
Michigan	16,234,000.80	7.328,316.91	612.6	2,807,574.86	1,334,319.27	115.3	13,614,403.05	6,276,891.49	389.2	309.916.00	113,210.57		3,072,208.76	Michigan
Minnesota	30,415,685.89	12,738,642.04	2721.2	1,605,828.93	748,422.07	176.7	8,420,751.61	3,475,400.00	687.8	844,036.79	52,800.00	48.5	446,347.89	Minnesota
Mississippi	10,292,285.79	4.988.702.73	803.4	1.037.377.99	517,960.94	76.8	8.275.985.70	4,131.737.19	434.5	866.500.44	428,250.19	68.5	768,163.95	Mississippi
Missouri	17,368,156.57	8.219,411.43	1118.9	1.765.817.26	852,178.33	85.7	26.543,802.25	10.764.686.97	788.7	1,854.369.22	145.005.53	42.9	398.306.74	Missouri
Montana	10,156,600.41	5,317,523.15	921.6	448,444.40	390,460.39	49.5	1,823,262.71	1,278,779.76	150.7	882,285.54	461,565.89	102.7	4,424,895.81	Montana
Nebraska	9,306,374.36	4.389.523.60	1570.6	1,112,300.53	537,102.43	99.6	8.779,538.78	4,358.961.31	880.9	2.594,699.03	1.268,245.81	228.6	2,493.273.95	Nebraska
Nevada	4,917,465.69	3,088,299.78	357.3	176,699.88	131.175.94	18.2	5,229,691.47	4.383,698.11	448.4	111,809.24	98,079.06	19.5	145.644.12	Nevada
New Hampshire	4,165,687.86	1,986.226.87	208.1				1.232,186.05	581,902.15	40.3	56,895.78	22,710.00	1.5	213,027.98	New Hampshire
New Jersey	11,961,357.45	3.820.679.99	219.1	568.121.43	131.911.06	8.1	10.212,826.36	3.529.508.12	83.5				50,612.83	New Jersey
New Mexico	8,717,999.18	4,914,070.61	1081.3	2.086,565.12	1,394.254.27	170.9	2.560,206.42	1.625.371.41	252.8	272,777.67	190.930.39	17.4	1.660,495.32	New Mexico
New York	28,597,76[illegible].67	12,229,076.53	831.5	2,744,698.59	1,066,085.79	67.9	31,735,025.75	10,375,109.05	641.2	8,859,600.00	2,222,502.50	147.5	4,506,256.13	New York
North Carolina	21,014,450.41	8.746,454.69	1119.8	2,066,128.35	867,034.98	62.1	7.708,632.34	3,340,365.95	193.2	1.449,872.60	596,457.86	37.2	458,338.63	North Carolina
North Dakota	10,829,263.82	5.268,930.47	1917.5	573,986.80	286.862.87	141.9	3.749,139.68	1.890.221.73	479.3	1.012.332.91	506.162.18	117.7	1,602,761.75	North Dakota
Ohio	41,572,252.81	15,244,993.93	1191.1	1,843,360.94	644,759.37	56.4	12,024,843.47	4,554,772.20	363.0	1,574,151.83	488,740.22	36.7	2,021,493.28	Ohio
Oklahoma	20,787,024.94	9,672.890.34	852.2	1.529,982.01	684,504.73	52.3	7,147.660.01	3.415,280.80	338.5	795.052.10	267.008.83	31.4	267,857.30	Oklahoma
Oregon	14,388,188.70	7,142.364.63	794.6	424.964.52	235,853.72	26.1	3,631,754.07	2.049.888.22	179.3	508.820.95	137,767.25	23.0	130.528.18	Oregon
Pennsylvania	43,054,835.19	16,222,023.97	850.3	1,834,714.33	513,286.76	32.5	35,159,598.89	9,353,875.14	612.6	3,935,957.11	1,344,278.66	111.8	558,396.47	Pennsylvania
Rhode Island	2,628,496.20	1,119.688.09	64.8				2,543,268.84	681.939.69	37.2				500,316.32	Rhode Island
South Carolina	11,163,347.84	5,121.267.54	1235.9	1.051.972.51	357.678.74	58.5	6.010.468.85	2.824,091.68	328.9	892,715.51	381.496.01	61.5	64,997.03	South Carolina
South Dakota	12,091,434.67	5,989.879.00	1447.9	1.220.287.86	585.884.32	141.9	6,871.535.15	3.272,429.89	972.2	169,162.07	44,213.78	28.2	52.185.01	South Dakota
Tennessee	13,789,140.98	6.732,079.77	497.9	2,555,128.53	1.261.638.03	84.4	9.707.053.23	4.327,533.01	338.4	386.669.75	149.846.57	17.6	1.191.074.62	Tennessee
Texas	54,120,970.83	21,057,940.12	3907.1	3.903,731.98	1.588.337.77	229.0	22,953.818.50	10.048,344.36	1354.9	4.093,076.77	1,818,327.52	309.6	1.666.564.23	Texas
Utah	6,259,159.41	3.818.836.91	423.1	132.178.99	99.880.35	5.5	3,224,624.68	2.193,626.23	255.5	23.746.72	17.054.73	1.0	841.129.78	Utah
Vermont	3,015,174.51	1.452.894.45	107.8	153.910.93	76.897.43	3.2	1.777,483.29	777,488.36	38.3	145.406.47	72.703.23	4.8	522,898.53	Vermont
Virginia	13,099,720.01	6.271.998.20	676.2	3.336.056.31	1.597,695.44	139.2	7,989,756.66	3.488,740.13	252.2	1.204.196.14	560.587.67	44.8	136.640.56	Virginia
Washington	13,352,504.18	6.117,211.87	526.7	1.315.032.75	550,338.64	64.1	3,080.756.36	1.427.900.00	98.5				920.245.49	Washington
West Virginia	7,343,200.86	3.230.293.33	326.7	252.664.30	123,013.89	12.8	6.780,995.84	2,722,663.90	179.1	981,982.41	387,090.29	35.2	115.513.59	West Virginia
Wisconsin	21,807,140.91	8.919,640.62	1451.7	107.355.47	53.526.17	6.6	5,681,321.94	2.794,796.59	277.6	484,386.26	238.519.50	35.7	3,562,070.12	Wisconsin
Wyoming	8,809,819.33	4.739,096.67	982.0	622,379,97	374,550.58	50.4	3,650,537.43	2,297,442.53	251.2	73,606.61	47,255.00	9.9	172,336.22	Wyoming
Hawaii							342,277.22	97,440.00	6.5				637,088.00	Hawaii
TOTALS	740,140,790.82	325,654,346.00	41898.3	53,648,350.79	24,493,628.57	2769.3	396,041,221.25	171,541,585.74	16717.4	45,943,009.77	16,921,235.75	2263.5	59,639,203.94	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 104,492,191.53 Federal aid $ 46,448,700.02 Miles 4,531.4

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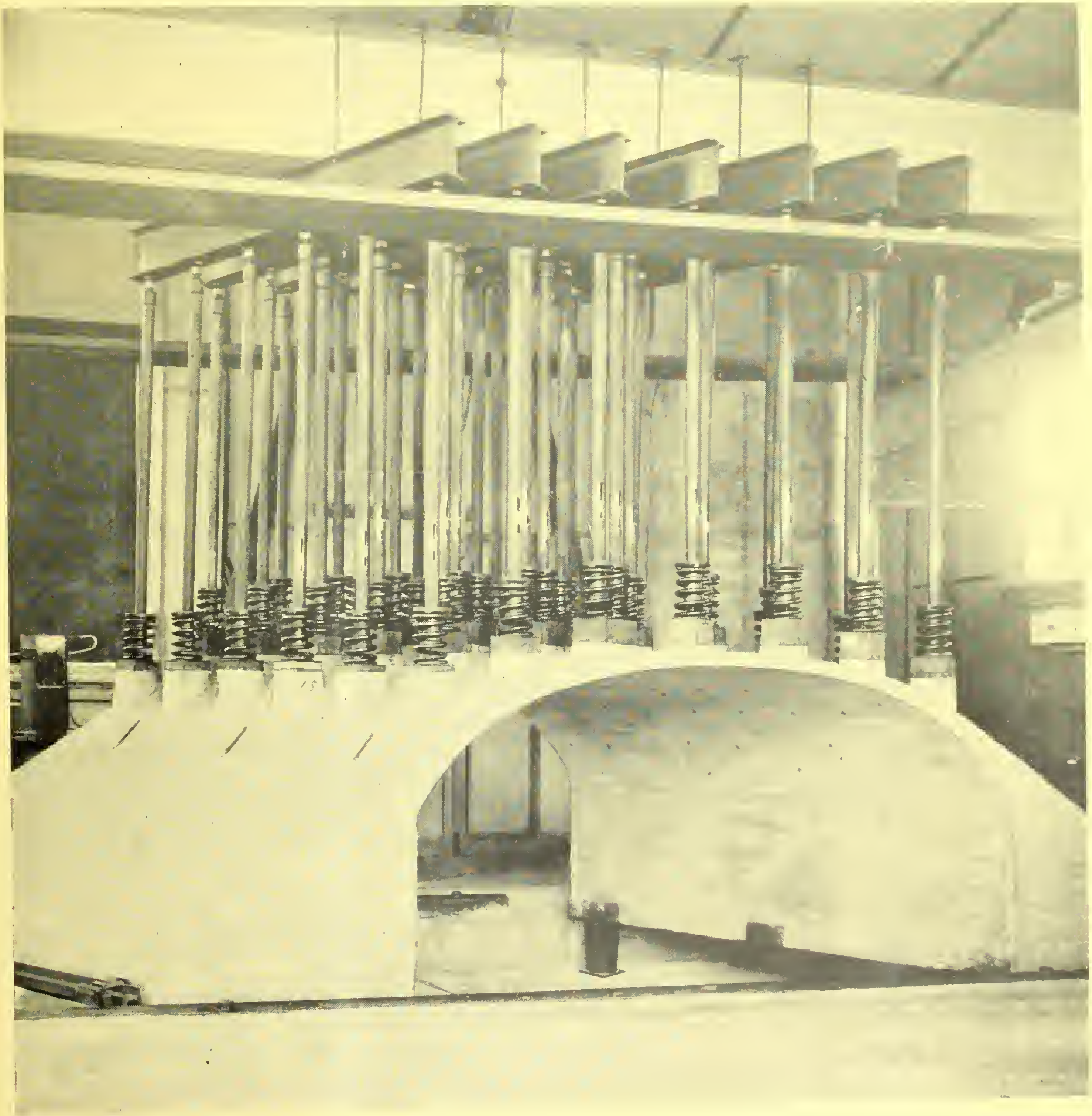
PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 6, NO. 9 ▽ NOVEMBER, 1925



Model Concrete Skew Arch Showing Method of Applying Test Loads

WASHINGTON : GOVERNMENT PRINTING OFFICE : 1925

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

VOL. 6, NO. 9 NOVEMBER, 1925 **H. S. FAIRBANK, Editor**

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PROGRESS REPORT OF SKEW ARCH TESTS

BY THE DIVISION OF TESTS, U. S. BUREAU OF PUBLIC ROADS

Reported by G. W. DAVIS, Associate Engineer of Tests

LACKING evidence to indicate the necessity for special design, early bridge designers assumed the stress distribution in skew arches to be similar to that in a symmetrical right arch. Rankine says:

> Skew arches are of figures derived from those of symmetrical arches by distortion in a horizontal plane. The elevation of the face of a skew arch, and every vertical section parallel to its face, being similar to the corresponding elevation and vertical section of a symmetrical arch, the forces which act in a vertical layer or rib of a skew arch with its abutments, are the same with those which act in an equally thick vertical layer of a symmetrical arch with its abutments, of the same dimensions and figure, and similarly and equally loaded.[1]

"The angle of skew, or obliquity," as defined by Rankine and still commonly accepted, "is the angle which the axis of the archway * * * makes with a perpendicular to the face of the arch. * * * The span of the archway, 'on the square,' as it is called (that is, the perpendicular distance between the abutments) is less than the span on the skew, or parallel to the face of the arch, in the ratio of the cosine of the obliquity to unity. It is the span on the skew which is equal to that of the corresponding symmetrical arch." [1]

Hool expressed a similar opinion, as follows: "Skew arches may be treated exactly as right arches, the span being taken parallel to the center line of roadway and not at right angles to the springing lines of the arch." [2]

This assumed theory of design was apparently discredited when, on May 14, 1903, a Monier arch of considerable size and extreme skew collapsed under a trial test at Bendigo, Australia, with loss of life. The failure was sudden and unexpected and unlike those which had been recorded in the cases of similar arches in Europe and elsewhere, in which signs of failure were visible long before the final collapse. The contractor, believing the failure to be due to faulty design, retained W. C. Kernot to determine the reasons for the collapse, and his report contains some valuable information. "Assuming the abutments to be immovable," said Mr. Kernot, "which must be very nearly correct, as they are small in height, massive in section, and founded on solid rock, and that the arch was perfectly and uniformly elastic, so that the stress in any given direction would be proportional to the change of dimensions, the following calculation was made. The arch was supposed to have been pressed down to a given small extent by its own weight, and the superincumbent load, and the consequent reduction in length of lines traced on its extrados in the positions *AB* and *AC* (fig. 1), were computed, the result being that the percentage of shortening in the direction *AC* was found to be almost exactly double that in the direction *AB* or in any line parallel to *AB*. Consequently, it followed that the heaviest stress in the arch was diagonally across from *A* to *C*, that the stress in the direction *AB* was only half as great, while that in the direction *BD* was found by an extension of the same calculation to be excessively small." [3]

An investigation of the failure confirmed these computations. It was found that the abutment at the obtuse angle of the arch "had been absolutely pulverized though of good, strong concrete, thus bearing witness to the great local intensification of stress." To overcome this unequal distribution of stress it was recommended that the mass of concrete in the abutment of the obtuse angle of the arch be increased, and that the arch ring be thickened. The thickening reommended was from one-fourth to one-half the thickness computed for a right arch corresponding to a variation in the skew from 20° to 50°.

The increased thickness was to diminish in all directions until it vanished at a distance equal to one-fourth of the skew span. Similar provisions were made for the abutments. The acute angles of the abutments were also replaced by right angles.

This failure added to the doubt which was already felt by some engineers as to the soundness of the theory of design which treated skew arches exactly as right arches except that the span was taken parallel to the faces of the arch and not at right angles to the springing line. As a result, a few tests have been made in recent years to determine the distribution of stresses in skew arches. C. R. Young[4] showed by means of india-rubber models that the generally accepted practice was in error. Tests made by C. H. Spofford, of the Massachusetts Institute of Technology, as well as a series of tests, using model concrete arches, carried on under the direction of John R. Chamberlin, deputy highway commissioner of Ohio, and Clyde T. Norris, of Ohio State University, by P. H. Ellman, and W. Hartline, all tended to show that the resultant pressures of skew arches fell near the obtuse angle of the arch.

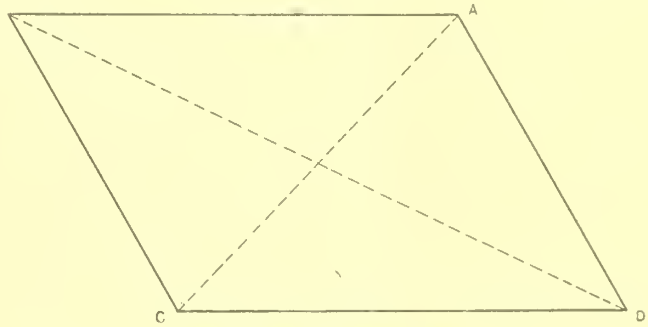


FIG. 1.—Plan of typical skew arch

BUREAU TESTS MADE NECESSARY BY SKEW ARCHES PROPOSED FOR FEDERAL-AID HIGHWAY SYSTEM

The lack of experimental data and the occasional necessity of using skew arches in peculiar locations on the Federal-aid highway system caused the Bureau of Public Roads to begin a series of tests in July, 1922. The information sought was the distribution and intensity of the abutment reactions and the amount of deformation of the arch ring along the springing line under uniform loads. It was hoped that the results would provide a more rational basis for design.

The results have been encouraging, but the experiments may not yet be regarded as complete. The tests have been made on model concrete arches, one-quarter size, with skews of 15°, 30°, 45°, and 60°. In all, the span parallel to the faces has been uniformly 7 feet; consequently the span at right angles to the springing line has been variable. Before it will be possible to formulate a mathematical basis for design, therefore, it will be necessary to make complete tests, (1) with the transverse width of the arch and the span at right angles to the springing lines constant and the skew variable, (2) with the transverse width of the arch and the skew constant and the span at right angles to the springing lines variable, and (3) with the span at right angles to the springing lines and the skew constant and the transverse width of the arch variable. All of these tests, moreover, should be made for various heights of rise.

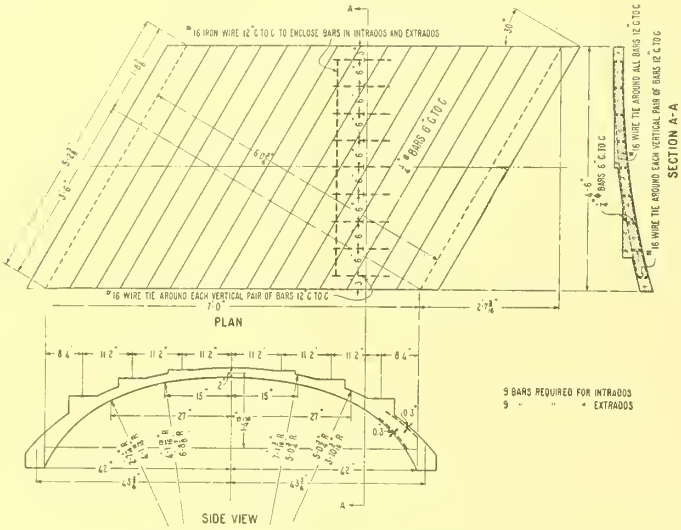
---

[1] A Manual of Civil Engineering, by W. J. M. Rankine, p. 429, sec. 295.

[2] Reinforced Concrete Construction, G. A. Hool, vol. III, p. 43, sec. 22.

[3] The Stresses in Skew Arches, by W. C. Kernot, Engineering News, vol. 49, no. 24, June 11, 1903, p. 529.

[4] University of Toronto, Toronto, Ontario, Canada.



#16 IRON WIRE 12" C TO C TO ENCLOSE BARS IN INTRADOS AND EXTRADOS
A
30°
½" # BARS 6" C TO C
6'-0¾"
5'-2⅞"
3'-6"
1'-8⅞"
#16 WIRE TIE AROUND EACH VERTICAL PAIR OF BARS 12" C TO C
7'-0"
2'-7⅜"
PLAN
4'-6"
½" # BARS 6" C TO C
#16 WIRE TIE AROUND ALL BARS 12" C TO C
#16 WIRE TIE AROUND EACH VERTICAL PAIR OF BARS 12" C TO C
SECTION A-A
9 BARS REQUIRED FOR INTRADOS
9 " " " EXTRADOS
8.4"
11.2"
15"
27"
42"
43¾"
0.3'
SIDE VIEW

Fig. 2.—Detailed sketch of 30° skew arch. The structural details and dimensions for the 15°, 45°, and 60° arches were the same with the exception of the span at right angles to the abutments and the oblique length of the abutments

TENTATIVE CONCLUSIONS DRAWN FROM BUREAU TESTS

As a result of the tests thus far made by the bureau the following tentatvie and general conclusions may be drawn.

1. The vertical reactions varied uniformly from a maximum at the obtuse angle of the arch to a minimum at the acute angle.
2. The increase of the vertical reaction at the obtuse angle of the arch varies directly as the angle of skew.
3. The resultant pressures of both the horizontal and vertical components move toward the obtuse angle of the arch with an increase in the skew.
4. The direction of the side thrust is toward the face of the arch at the acute angle and the intensity increases with the degree of skew.

DESCRIPTION OF THE TESTS

The mechanical difficulties of the tests resolved themselves into the development of a method of applying uniform loads to the model arches and some means of accurately measuring the resultant vertical and horizontal reactions without movement of the arch under load. A preliminary test was made during the summer of 1923 on a 30° skew arch with abutments, using Kreuger cells to measure the abutment reactions and Berry strain gauges to measure the stresses in the arch ring. The results were unsatisfactory due to the difficulty of obtaining complete sets of readings on a number of Kreuger cells which was caused principally by poor impressions on some of the carbon films, and because of the variation in the Berry strain gauge readings due to the personal equation. These results were finally discarded and the successful tests were made as follows:

The model arches tested, which were one-fourth full size, had skews of 15°, 30°, 45°, and 60°. All were 7 feet in span as measured parallel to the faces, 1.4 feet in rise, 4½ feet face to face, and 2 inches thick at the crown. They were made of mortar mixed of 1 part Portland cement and 3 parts Potomac River sand with a slump of 3 to 4 inches. The reinforcement consisted of eighteen ¼-inch deformed bars placed in vertical pairs parallel to the faces of the arch, 6 inches center to center, with ties, both horizontal and vertical around each pair, made of No. 16 iron wire spaced 12 inches center to center. (Fig. 2.)

All arches were cut off at the springing line and a base or abutment with a squared section 4 inches on a side and reinforced with two ½-inch deformed bars was cast as an integral part of the arch rings. Steel bearing plates were cast in place in the base at those points at which reactions were to be measured.

UNIFORM LOAD APPLIED BY SPRINGS

Uniform loads were applied on 42 symmetrically placed loading points on the arch ring by compressing helical springs 5 inches in diameter and 7 inches high, of a uniform compressive strength (within 2 per cent) of 915 pounds per inch. Forty-two vertical struts of 1½-inch pipe bolted at their upper ends to a cribbing of I-beams took the vertical thrust. This cribbing was supported by a heavy beam directly over the experimental arch, and the carrying beam was, in turn, supported at each end by a 2-inch bolt which engaged a steel yoke bolted to the webs of two upright channel irons anchored to the floor and separated from each other by a space sufficient to clear the overhead beam. By this arrangement the entire cribbing could be raised or lowered by means of nuts running on the supporting 2-inch bolts.

Approximately the entire desired load was applied simultaneously by lowering the steel cribbing, thus doing away with possible cross stresses which might have been introduced had the load been applied by screwing down the springs one by one. Final fine adjustments were made by individual screw jacks at the top of each vertical strut. These jacks consisted of 1-inch cap screws working through a nut forced into a coupling on the upper end of each pipe. The compression of each spring was measured by a fixed scale and a moving vernier actuated by a plunger inside of the strut. As the plungers rested at the center of the springs the compression measured was the mean compression of the spring. Scales and verniers read to hundredths of an inch or the nearest 10 pounds of load.

In the measurement of abutment reactions, procedure was as follows: The entire arch was surrounded by a rigid frame of concrete and steel angles bolted to the concrete floor. One side of the arch was fixed, being supported at three points on blocking and a ¾-inch steel ball. Horizontal thrusts at the fixed end were taken up by blocking and spherical bearing blocks placed between the rigid frame and the arch. At the other end measuring devices were placed to take both vertical and horizontal reactions in positions symmetrical with the fixed supports and blocking at the anchored end. (Fig. 3.)

Measuring devices for determining the amount of reactions consisted of soil pressure cells actuated by 16 to 1 lever arms. These lever arms consisted of a length of 25-pound rail resting on knife edges, the one at the load end resting on the floor and the other on the center of a soil pressure cell. These knife edges were 32 inches on centers. The load was applied through a ¾-inch ball and clock-spring knife edges at a point 2 inches from the fixed knife edge thus giving a reduction of 16 to 1. The area of diaphragm in the soil pressure cell being 10 square inches, it was possible to balance a 1,000-pound load by approximately a 6-pound air pressure.

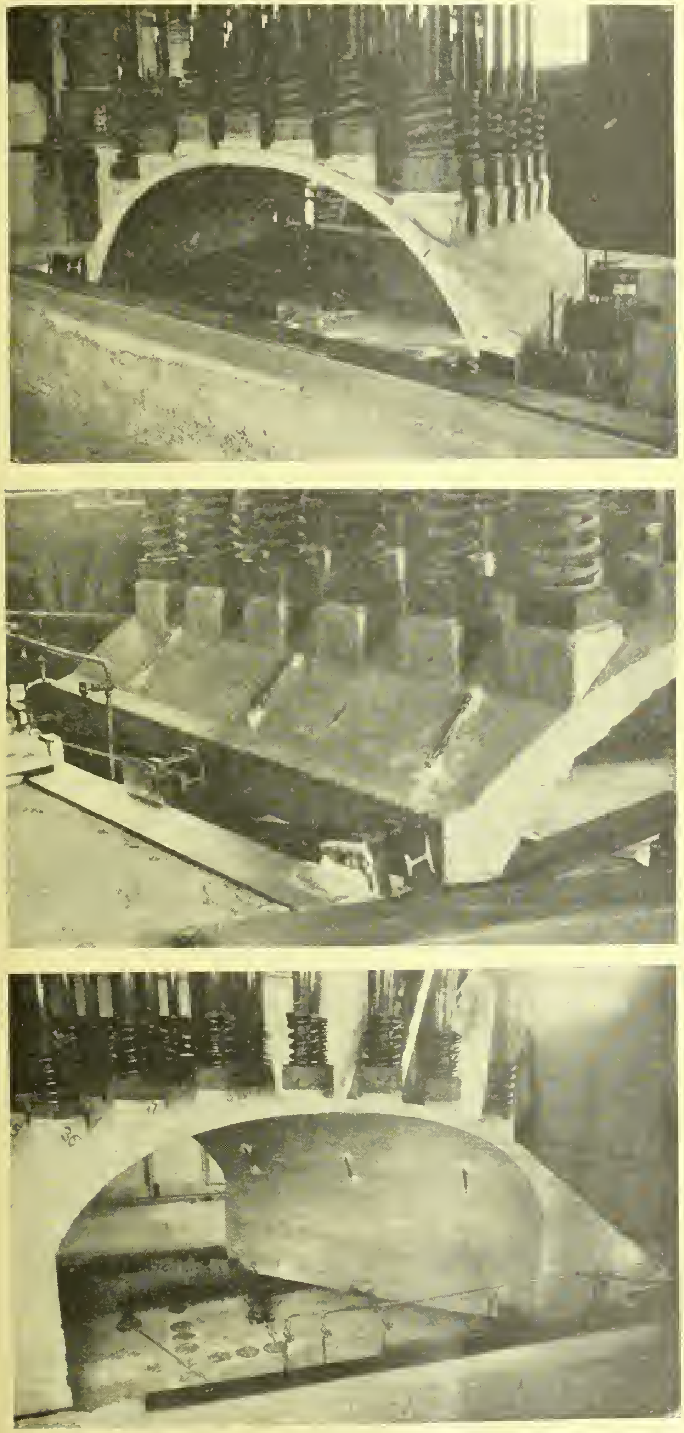


Fig. 3.—*Upper.*—Showing rigid frame surrounding an arch and blocking and spherical bearing blocks to take longitudinal thrust at fixed end of arch
Middle.—Showing graphic strain gages on extrados and levers for measuring horizontal thrusts and vertical reactions
Lower.—Showing graphic strain gages on intrados and levers and soil pressure gages for measuring vertical reactions

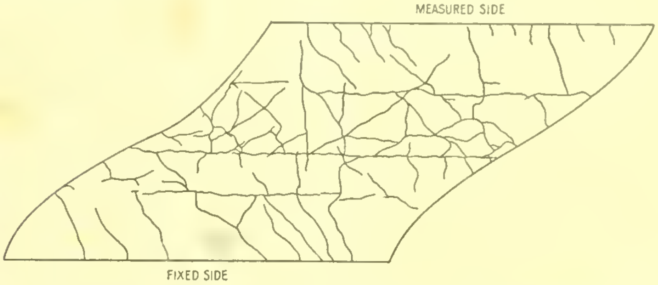


Fig. 4.—The intrados of the 60° skew arch showing the cracks which developed under final load of 60,000 pounds

Each soil pressure cell with its lever arm and knife edges was calibrated by placing it under the head of a testing machine under known loads. In place of the conventional pressure gauges used with the pressure cells for measuring soil pressures, a sealed monometer tube or mercury column was used and the readings in millimeters of the height of mercury column for each known load were recorded. Expansion of the air and mercury was controlled and the column brought back to a known zero by means of a jacket of ice water and the circulation was maintained by means of a rubber bulb. Several sets of readings were taken for each cell, the variation being negligible. The movement of the lever over the cell was found to be only 0.0004 inch, and the ratio of the lever arms being 16 to 1, the movement at the load point was obviously so small that the arch could be considered to remain stationary, which is one of the conditions sought in the tests.

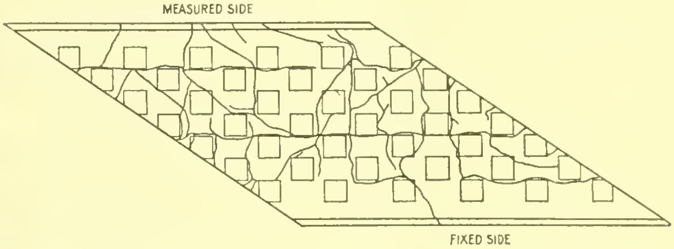


Fig. 5.—The extrados of the 60° skew arch showing the cracks which developed under final load of 60,000 pounds

The deformations of the arch ring were measured by means of graphic strain gauges mounted at three points on the intrados and three points on the extrados on the measured side of the arch. These gauges were placed 6½ inches above the springing line, one at the center and one on each side midway between the two outer loading points on each side and near the obtuse and acute angles respectively.

TEST OF SIXTY-DEGREE SKEW ARCH

The 60° skew arch was poured April 21, 1923, and loaded to destruction December 11 the same year, 235 days after pouring. The total applied load was increased by increments of 4,200 pounds, equal to 100 pounds on each spring, and the arch failed under a total load of 25.2 tons, equal to a spring load of 1,200 pounds. At each application of load up to the spring load of 500 pounds, the vertical reactions and horizontal thrusts at the springing line were measured at the obtuse and acute angles and at the center. Beyond the 500-pound spring load they were not measured because they exceded the capacity of the knife edges and measuring devices. The nature of the failure is shown in Figures 4, and 5.

Six test cylinders made at the time the arch was poured showed at the end of 125 days a mean breaking strength of 4,235 pounds per square inch and an average modulus of elasticity of 3,370,000.

The mean vertical reactions measured at the three points are shown in Table 1, and in graphical form in Figure 6, from which it will be seen that, in each case, they vary uniformly from a maximum at the obtuse angle to a minimum at the acute angle.

TABLE 1.—*Mean vertical reactions at three points of the 60° skew arch*

Theoretical applied load on one-half of arch	Mean measured vertical reactions due to applied load								Variation of measured reaction from theoretical applied load
	Obtuse angle		Center		Acute angle		Total		
Pounds	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Per cent*
2,100	1,365	70	550	28	40	2	1,955	100	6.9
4,200	2,880	69	1,225	29	65	2	4,170	100	.7
6,300	4,360	68	1,945	30	110	2	6,415	100	1.8
8,400	5,940	67	2,820	32	130	1	8,890	100	5.8
10,500	7,780	69	3,470	31	80	--------	11,330	100	7.9
Mean	--------	69	--------	30	--------	1	--------	100	----------

The mean horizontal thrusts are shown in Table 2 and also in Figure 6, from which it will be seen that the average thrust at the obtuse angle is approximately 63 per cent of the live load applied, while at center it is only approximately 4 per cent. The thrust at the acute angle decreases from 2.3 per cent of the live load under a theoretical load of 2,100 pounds to a negative thrust equal to 0.3 per cent of the live load under a theoretical load of 10,500 pounds.

TABLE 2.—*Mean horizontal thrusts at three points of the 60° skew arch*

Theoretical applied load on one-half of arch	Measured vertical reaction	Mean measured horizontal thrusts due to applied load							
		Obtuse angle		Center		Acute angle		Total	
		Thrust	Relation to measured reaction	Thrust	Relation to measured reaction	Thrust	Relation to measured reaction	Thrust	Relation to measured reaction
Pounds	*Pounds*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*
2,100	1,955	1,260	64.4	80	4.1	45	2.3	1,385	70.8
4,200	4,170	2,610	63.0	175	4.0	60	1.4	2,845	68.4
6,300	6,415	3,775	59.0	255	4.0	25	.4	4,055	63.4
8,400	8,890	5,300	59.6	455	5.1	−5	--------	5,750	64.7
10,500	11,330	7,665	67.7	560	4.9	−30	−.3	8,195	72.3
Mean	--------	--------	62.7	-------	4.4	--------	.8	--------	67.9

The variation of the total measured reaction from the theoretical applied load as shown in Table 1 is probably due to inaccuracy in the application of the load rather than to errors in the measured values.

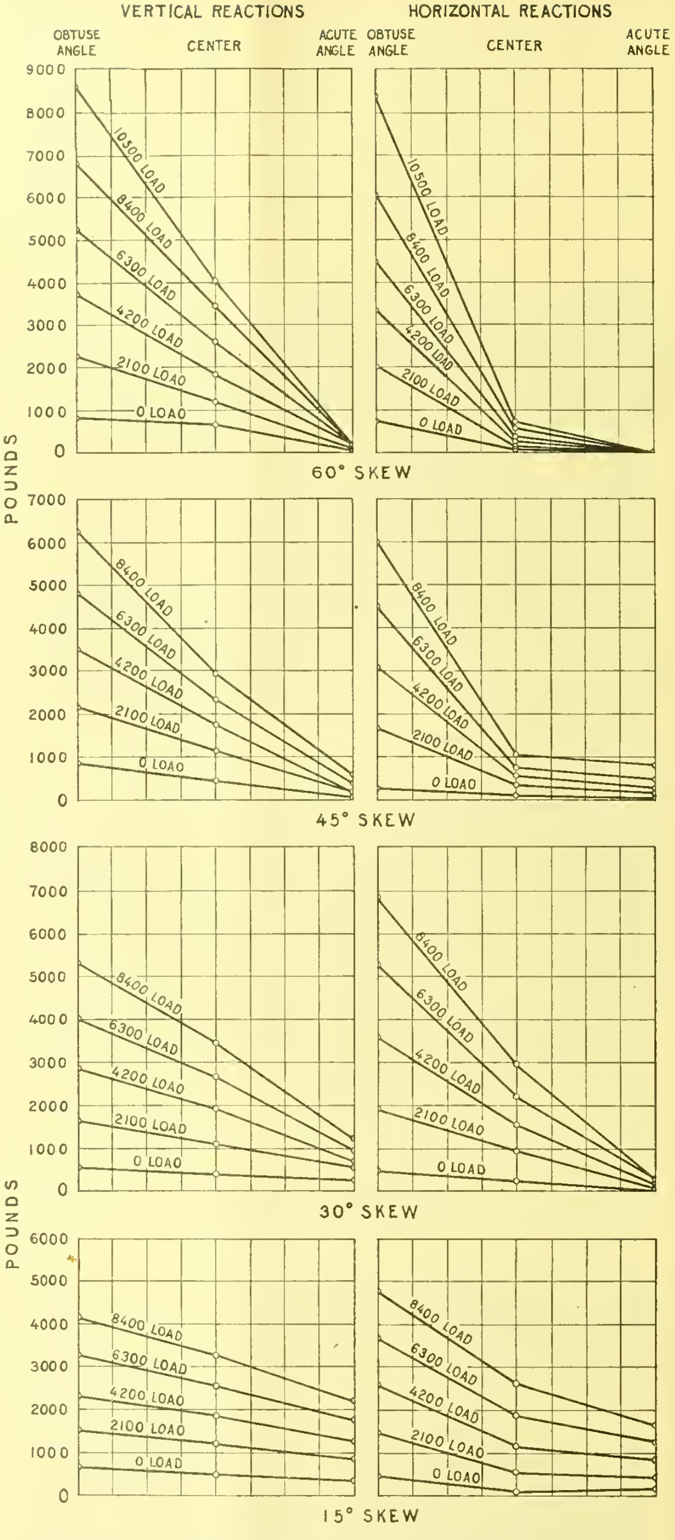


FIG. 6.—The amount of the vertical and horizontal reactions at the obtuse angle, center, and acute angle of the 15°, 30°, 45°, and 60° skew arches with no applied load and applied loads at increments of 2,100 pounds

This is apparent from the close agreement in the measured values themselves as indicated in Table 3.

The strain-gauge readings on this arch were erratic. It has since been found that this was due to the turning of the gauges in the bearing points when the smoked-glass slides of the strain gauges were moved up to a new position. Although the recorded deformations were not consistent for equal increments of load, the

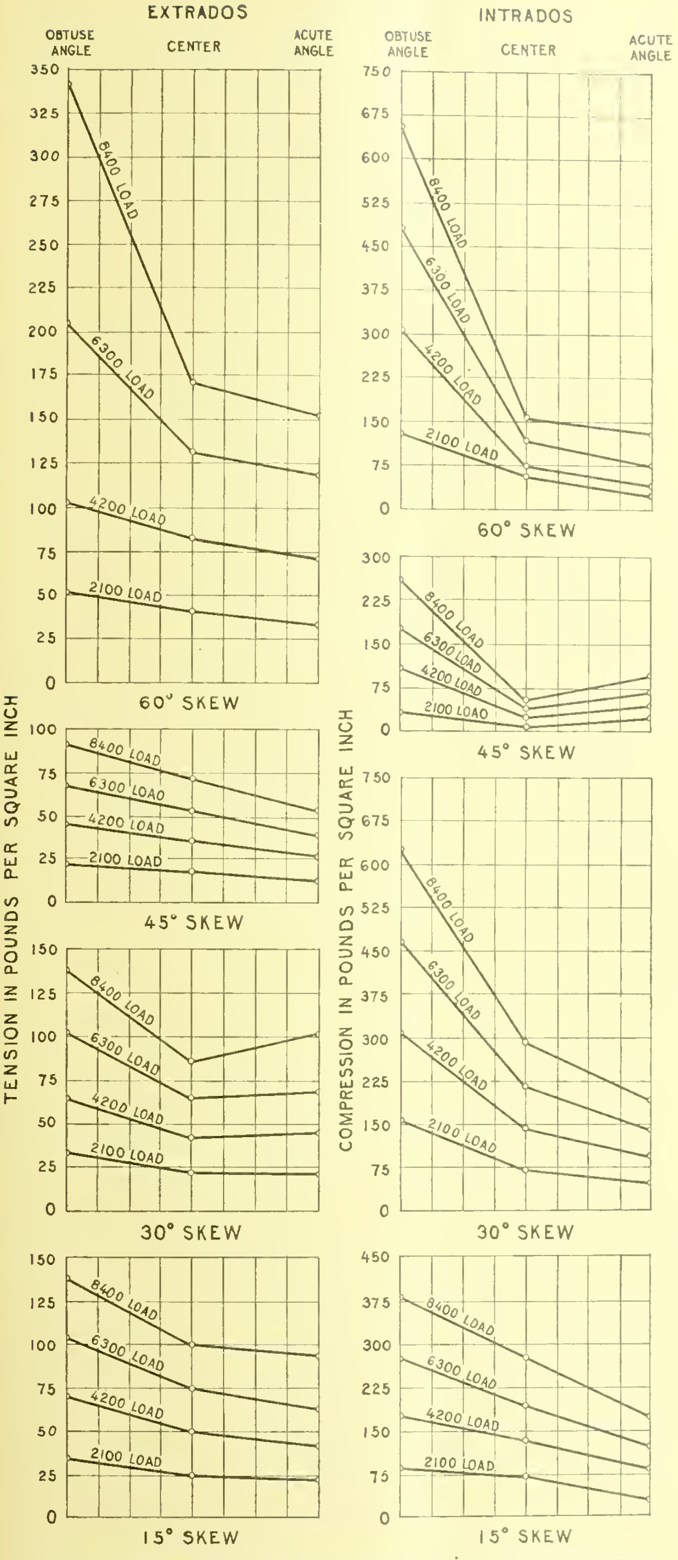


Fig. 7.—Stresses computed from strain-gage readings on the extrados and intrados of the 15°, 30°, 45°, and 60° skew arches for several increments of load

relative stresses checked the reactions as recorded, from four to five times the deformation shown at the acute angle being recorded at the obtuse angle. The stress indicated in the intrados at the obtuse angle under a total load of 16,800 pounds was approximately 650 pounds per square inch as against 130 pounds per square inch at the acute angle. (See fig. 7.)

Table 3.—*Variation in measurements of vertical reactions and horizontal thrusts in the 60° skew arch*

Theoretical applied load on one-half of arch	Vertical reactions		Horizontal thrusts	
	Number of readings	Maximum variation from mean sum	Number of readings	Maximum variation from mean sum
Pounds		*Per cent*		*Per cent*
2, 100	13	2.3	13	8
4, 200	13	2.0	13	9
6, 300	8	.5	8	3
8, 400	6	.5	6	7
10, 500	4	2.0	4	2

TEST OF THIRTY-DEGREE SKEW ARCH

The 30° skew arch was poured December 15, 1923. The loading and set-up for the test, the points at which the reactions were read and the relative positions of the strain gauges were the same as in the test of the 60° arch. In this case, however, the rotation of the strain gauges was prevented by placing wooden wedges between the body of the gauge and the arch.

It was shown by the tests of the 60° skew that the thrust is diverted toward the obtuse angle resulting in tortion or a tendency of the arch to swing on the abutments. This tendency of the arch to swing or rotate—causing the acute angle to pull away from the measuring device—was so great under all but small loads that ribbon knife-edges were thrown out of the vertical position and sheared. To prevent the breaking of the knife-edges, it was found necessary to counteract this swinging tendency by fixed wedges placed against the face of the arch at the acute angle on the anchored end with a pressure cell and a 2 to 1 lever between the face and rigid framework at the acute angle at the measured end.

Six sets of readings for the vertical reactions, horizontal thrusts, and side swings were taken with unit spring loads varying from 100 to 400 pounds. The mean vertical reactions, as measured at three points, are shown in Table 4, from which it is apparent that for the 30° skew 54 per cent of the load applied is transmitted to the obtuse angle of the arch, 34 per cent to the center, and 12 per cent to the acute angle. By reference to Figure 6 it will be seen that in this case, as in the 60° arch, the vertical reactions vary uniformly from a maximum at the obtuse angle to a minimum at the acute angle.

Table 4.—*Mean vertical reactions at three points of the 30° skew arch*

Theoretical applied load on one-half of arch	Mean measured vertical reactions due to applied load								Variation of measured reaction from theoretical applied load
	Obtuse angle		Center		Acute angle		Total		
Pounds	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Per cent*
2, 100	1, 100	54	685	33	260	13	2, 045	100	2.6
4, 200	2, 305	55	1, 418	34	445	11	4, 168	100	.7
6, 300	3, 455	54	2, 245	35	690	11	6, 390	100	1.4
8, 400	4, 745	54	3, 075	35	935	11	8, 755	100	4.2
Mean	--------	54	--------	34	--------	12	--------	100	----------

The mean horizontal thrusts for six sets of readings are given in Table 5, and the mean side thrusts in Table 6. The variation in the reaction measurements are shown in Table 7. It is evident from the latter table that, in this test as in the test of the 60° skew arch, the principal inaccuracy is in the application of the load rather than the measurement of the reactions. The large percentage of variation in the side thrust under the first three loads is due to the small pressures. It should be noted that the fourth reading is more stable.

Table 5.—*Mean horizontal thrusts at three points of the 30° skew arch*

Theoretical applied load on one-half of arch	Measured vertical reaction	Mean measured horizontal thrusts due to applied load							
		Obtuse angle		Center		Acute angle		Total	
		Thrust	Relation to measured reaction	Thrust	Relation to measured reaction	Thrust	Relation to measured reaction	Thrust	Relation to measured reaction
Pounds	*Pounds*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*
2,100	2,045	1,425	70	705	34	35	1.7	2,165	105.7
4,200	4,168	3,105	74	1,300	31	75	1.8	4,480	106.8
6,300	6,390	4,775	75	1,950	30	125	2.0	6,850	107.0
8,400	8,755	6,335	72	2,710	31	205	2.3	9,250	105.3
Mean	--------	--------	72.8	-------	31.5	--------	1.9	--------	106.2

Table 6.—*Mean side thrusts toward face of 30° skew arch at the acute angle*

Theoretical applied load on one-half of arch	Measured vertical reaction	Mean measured side thrust due to applied load	
		Side thrust	Relation to measured reaction
Pounds	*Pounds*	*Pounds*	*Per cent*
2,100	2,045	85	4
4,200	4,168	295	7
6,300	6,390	510	8
8,400	8,755	785	9
Mean	----------	----------	7

Table 7.—*Variation in measurements of reactions on the 30° skew arch*

Theoretical applied load on one-half of arch	Maximum variation of any sum from mean sum of reactions		
	Vertical reaction	Horizontal thrust	Side thrust
Pounds	*Per cent*	*Per cent*	*Per cent*
2,100	4	5	50
4,200	3	2	60
6,300	3	5	20
8,400	3	1	1

The strain-gauge readings on this arch were consistent for equal loads, undoubtedly due to the wedging of the gauges which prevented the rotation of the gauge points. Tension was found to exist at all three points measured on the extrados with a maximum at the obtuse angle of 138 pounds per square inch under a total load of 16,800 pounds. All three points on the intrados were found to be in compression, with a maximum at the obtuse angle and a minimum at the acute angle. The maximum compression developed under a total load of 16,800 pounds was 630 pounds per square inch. Figure 7 shows the mean values for various loads. Six test cylinders showed a breaking strength of 5,240 pounds per square inch and a mean modulus of elasticity of 3,813,000 at the end of 123 days.

TEST OF FORTY-FIVE-DEGREE SKEW ARCH

The 45° skew was poured April 24, 1924. Loading set-up and relative points at which reactions were measured and gauges placed were the same as in the 30° and 60° skews previously described.

Thirteen sets of readings for vertical reactions, horizontal thrusts, and side thrusts were taken with unit loads varying from 100 to 400 pounds per spring. These are shown respectively in Tables 8, 9, and 10, which are similar to the corresponding tables for other tests. The accuracy and consistency of the measurements is indicated by Table 11.

Table 8.—*Mean vertical reactions at three points of the 45° skew arch*

Theoretical applied load on one-half of arch	Mean measured vertical reactions due to applied load								Variation of measured reaction from theoretical applied load
	Obtuse angle		Center		Acute angle		Total		
Pound	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Per cent*
2,100	1,305	65.0	680	34.0	25	1.0	2,010	100	4.3
4,200	2,660	65.6	1,290	31.8	105	2.6	4,055	100	3.4
6,300	3,960	64.4	1,920	31.2	275	4.4	6,155	100	2.3
8,400	5,365	64.0	2,510	30.0	505	6.0	8,380	100	.2
Mean	--------	65.0	--------	32.0	--------	3.0	--------	100	----------

Table 9.—*Mean horizontal thrusts at three points of the 45° skew arch*

Theoretical applied load on one-half of arch	Measured vertical reaction	Mean measured horizontal thrusts due to applied load							
		Obtuse angle		Center		Acute angle		Total	
		Thrust	Relation to measured reaction	Thrust	Relation to measured reaction	Thrust	Relation to measured reaction	Thrust	Relation to measured reaction
Pounds	*Pounds*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*
2,100	2,010	1,375	68.4	260	12.9	65	3.2	1,700	84.5
4,200	4,055	2,830	69.8	515	12.7	215	5.3	3,560	87.8
6,300	6,155	4,240	68.9	730	11.8	455	7.4	5,425	88.1
8,400	8,380	5,730	68.4	995	11.9	695	8.3	7,060	88.6
Mean	--------	--------	68.9	-------	12.3	--------	6.1	--------	87.2

Table 10.—*Mean side thrusts toward face of 45° skew arch at the acute angle*

Theoretical applied load on one-half of arch	Measured vertical reaction	Mean measured side thrust due to applied load	
		Side thrust	Relation to measured reaction
Pounds	*Pounds*	*Pounds*	*Per cent*
2,100	2,010	175	9
4,200	4,055	410	10
6,300	6,155	680	11
8,400	8,380	905	11
Mean	----------	----------	10.2

TABLE 11.—*Variation in measurements of reactions on the 45° skew arch*

Theoretical applied load on one-half of arch	Maximum variation of any sum from mean sum of reactions		
	Vertical reaction	Horizontal thrust	Side thrust
Pounds	*Per cent*	*Per cent*	*Per cent*
2,100	3.4	7.2	15
4,200	3.7	8.5	14
6,300	3.4	7.3	10
8,400	1.2	8.3	9

Test cylinders made at the time the arch was poured showed a mean breaking strength of 4,885 pounds per square inch at 90 days and a mean modulus of elasticity of 3,309,000.

By the strain-gauge readings tension was found to exist at all points measured on the extrados with a maximum at the obtuse angle of 92 pounds per square inch under a load of 16,800 pounds on the entire arch. Compression existed at all three points on the intrados with a maximum of 260 pounds per square inch at the obtuse angle under a total load of 16,800 pounds. The strain-gauge readings were consistent and showed a distribution similar to the vertical and horizontal reactions.

TEST OF FIFTEEN-DEGREE SKEW ARCH

The 15° arch was poured on September 22, 1924. The set-up of loading and measuring devices as well as the location of the strain gauges was the same as in the previous cases.

Eleven sets of readings were taken for vertical reactions, horizontal thrusts, and side thrusts with unit loads per spring from 100 to 400 pounds in 100-pound increments. These are shown respectively in Tables 12, 13, and 14, and the accuracy and consistency of the measurements is indicated by Table 15.

Test cylinders at the end of 80 days showed a mean breaking strength of 5,060 pounds per square inch and a mean modulus of elasticity of 3,151,400.

TABLE 12.—*Mean vertical reactions at three points of the 15° skew arch*

Theoretical applied load on one-half of arch	Mean measured vertical reactions due to applied load								Variation of measured reaction from theoretical applied load
	Obtuse angle		Center		Acute angle		Total		
Pound	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Per cent*
2,100	845	44.2	660	34.4	410	21.4	1,915	100	8.8
4,200	1,765	44.3	1,355	33.9	870	21.8	3,990	100	5.0
6,300	2,650	43.9	2,070	34.3	1,310	21.8	6,030	100	4.3
8,400	3,510	43.2	2,850	35.1	1,770	21.7	8,130	100	3.2
Mean		43.9		34.4		21.7		100	

TABLE 13.—*Mean horizontal thrusts at three points of the 15° skew arch*

Theoretical applied load on one-half of arch	Mean measured horizontal thrusts due to applied load								
	Measured vertical reaction	Obtuse angle		Center		Acute angle		Total	
		Thrust	Relation to measured reaction	Thrust	Relation to measured reaction	Thrust	Relation to measured reaction	Thrust	Relation to measured reaction
Pounds	*Pounds*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cent*	*Pounds*	*Per cen*
2,100	1,915	980	49.5	490	24.8	330	16.7	1,800	91.0
4,200	3,990	2,070	51.9	1,100	27.6	625	15.6	3,795	95.1
6,300	6,030	3,200	53.1	1,780	29.5	1,005	16.7	5,985	99.3
8,400	8,130	4,330	53.3	2,540	31.3	1,495	18.4	8,365	103.0
Mean			51.9		28.3		16.9		97.1

Strain-gauge readings gave practically a straight line distribution under all loads and showed tension on the extrados and compression on the intrados. Maxima were found at the obtuse angle in both cases and showed a maximum tension of 138 pounds per square inch and a maximum compression of 378 pounds per square inch under a total load of 16,800 pounds on the entire arch. It is not believed that a direct comparison of stresses as measured in the arches of various skews would be of value, due to varying temperatures

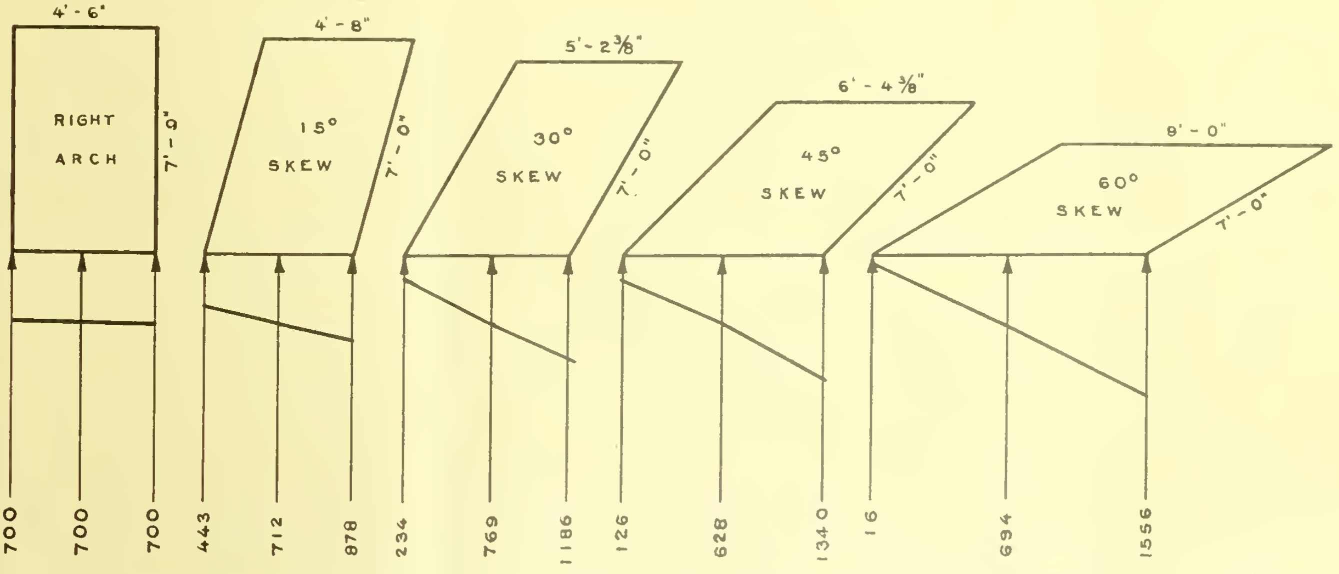


FIG. 8.—Mean vertical reactions of the 15°, 30°, 45°, and 60° skew arches and a right arch. The figures represent the mean of reactions corresponding to an increment of load of 2,100 pounds on one-half of the arch

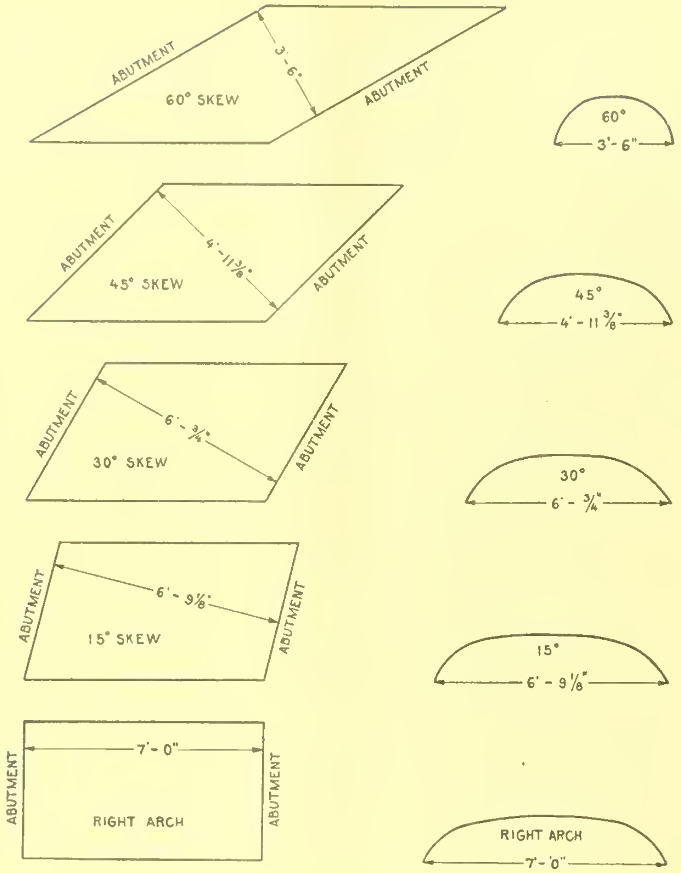


Fig. 9.—Plan of skew arches and dimensions of arch rings at right angles to the abutments. The shortening of the arch ring increases with the degree of skew

existing during the periods in which tests were made. The distribution of the recorded stresses is, however, a check on the recorded reactions for the various arches. Figure 7 shows the strain-gauge readings for the various arches.

Table 14.—*Mean side thrusts toward face of 15° skew arch at the acute angle*

Theoretical applied load on one-half of arch	Measured vertical reaction	Mean measured side thrust due to applied load	
		Side thrust	Relation to measured reaction
Pounds	*Pounds*	*Pounds*	*Per cent*
2,100	1,915	130	7
4,200	3,990	295	7
6,300	6,030	475	8
8,400	8,130	690	8
Mean			7.5

Table 15.—*Variation in measurements of reactions on the 15° skew arch*

Theoretical applied load on one-half of arch	Maximum variation of any sum from mean sum of reactions		
	Vertical reaction	Horizontal thrust	Side thrust
Pounds	*Per cent*	*Per cent*	*Per cent*
2,100	4.3	7.6	35.0
4,200	1.0	7.0	22.0
6,300	2.3	6.9	29.0
8,400	3.0	5.0	12.5

COMPARISON OF THE TEST RESULTS ON ALL ARCHES

Vertical reactions.—A comparison of the vertical reactions for the right arch and the 15°, 30°, 45°, and 60° skew arches is shown in Figure 8. Values used for the right arch are theoretical values; those for the skew arches are measured values. Vertical ordinates as plotted are mean increments due to successive loads of 100 pounds per spring or an increment in load of 2,100 pounds on one-half of the arch. Comparisons are made at the obtuse angle, the center, and at the acute angle.

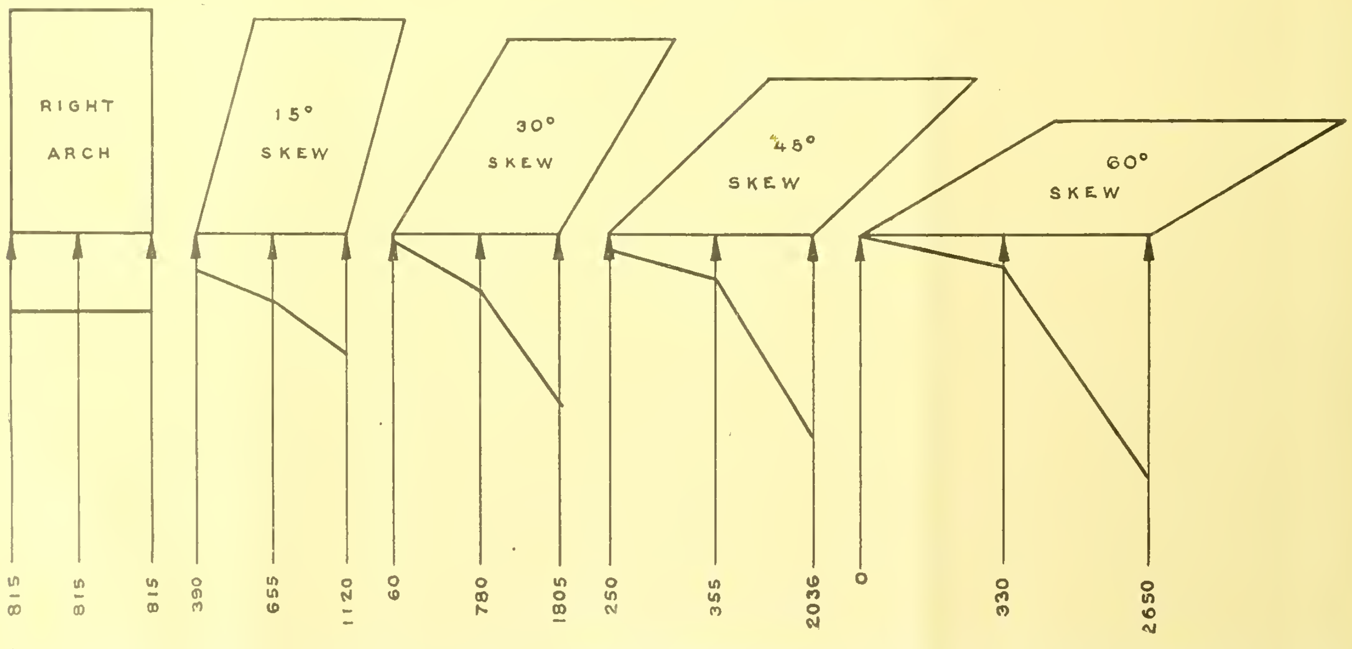


Fig. 10.—Mean horizontal reactions of the 15°, 30°, 5°, and 60° skew arches. The figures represent the mean of reactions corresponding to an increment of load of 2,100 pounds on one-half of the arch

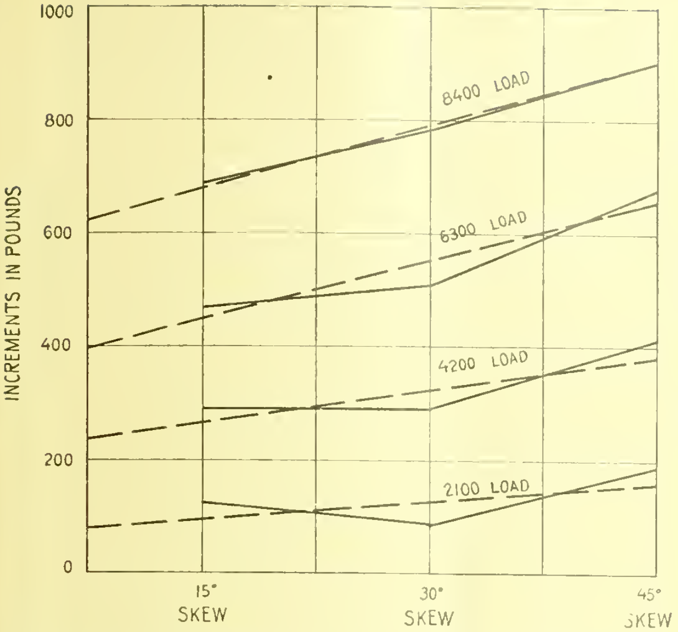


FIG. 11.—Mean side thrusts toward the face of the arch at the acute angle of the 15°, 30°, 45°, and 60° skew arches. The figures represent the mean of reactions corresponding to an increment of load of 2,100 pounds on one-half of the arch

From an inspection of Figure 8, it will be seen that the vertical reactions increase at the obtuse angle from one-third of the load applied for the right arch to two-thirds of the load applied for the 60° skew and that this increase varies uniformly as the angle of skew. At the center point the vertical reactions remain constant at one-third the load applied for the right arch and all the skew arches. At the acute angle vertical reactions decrease from one-third of the load applied for a right arch to practically zero (1 per cent) for a 60° skew, this decrease varying uniformly with the angle of skew. It should be noted that the sum of the vertical reactions for any given arch equals the load applied within allowable limits (less than 6 per cent).

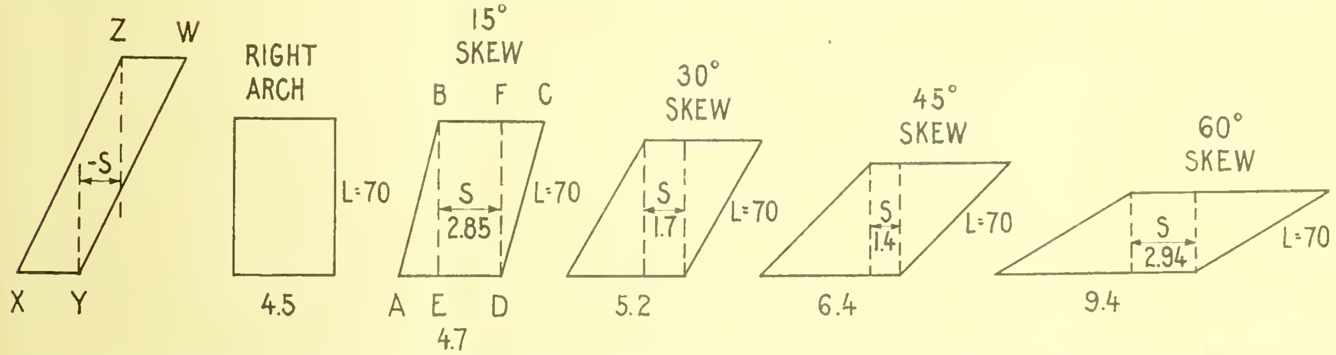


FIG. 12.—Sections showing variation in abutment length and span "on the square" for right and skew arches of various skews with uniform width normal to the faces of the arch. The width of sections of right arch included within the measured arches is indicated by S. Section X Y W Z shows a type of arch in which there is no right section. This type was not included in the tests

Horizontal reactions.—The model arches used in these tests were all built using the same arch ring with a 7 foot span. As the degree of skew was increased, the actual barrel or span of the arch normal to the abutments was reduced to an amount equal to the span of the right arch times the cosine of the angle of skew, or in the case of the 60° skew, a shortening of 50 per cent. (See fig. 9.) Theoretical thrusts for right arches of these shortened spans under the same loading vary from theoretical thrusts for a right arch of 7-foot span directly as the cosine of the angle of skew causing the shortening. In order that a direct comparison of the horizontal thrusts might be made, the measured thrusts were corrected by dividing by a factor equal to the cosine of the angle of skew to bring all the readings to the basis of an arch with a barrel span of 7 feet, equivalent to that of the right arch. The comparative thrusts for the various skews are shown in Figure 10. It will be noted that the same general relationship exists as is shown for the vertical reactions.

Side thrusts.—A comparison of the side thrusts for those arches in which this force was measured is shown in Figure 11. No measurements were taken for the 60° skew. The wide variation in the recorded results for the other skews makes any comparison approximate, although in a general way it is apparent from Figure 11 that the side thrust increases directly with the corresponding increase in the angle of skew. It is also interesting that the differences in the average increments of side thrust due to 2,100-pound increments of load varies from 25 pounds between 15° and 30° to 30 pounds between 30° and 45°. This indicates that the side thrust increases almost directly with the angle of skew. The measurements made for the small loads are unreliable because of the difficulty of making accurate measurements of small loads.

Strain-gauge readings.—There are some obvious deductions which may be made from Figure 7, although the strain-gauge readings are not measures of the comparative stresses existing in the arches because of the wide variations in the readings caused by changes in temperature. The stresses are greatest in the arch ring adjacent to the obtuse angle of the arch and decrease as the acute angle is approached. This furnishes a rough check on the measurements of the horizontal and vertical reactions.

TESTS MUST BE CONTINUED

In comparing the results for the different degrees of skew it must be remembered that the transverse width of the arch (4 feet 6 inches) and the length parallel to the face of the arch (7 feet) have remained constant, but the span at right angles to the springing lines has varied. This prevents an accurate comparison of results. For example, consider Figure 12. For each one of the arches lines have been drawn normal to the springing lines from the obtuse angles. This results in a section of right arch for each test case. The distance S

(Continued on page 202)

EFFICIENCY IN CONCRETE ROAD CONSTRUCTION

A REPORT OF OBSERVATIONS MADE ON GOING PROJECTS BY THE DIVISION OF CONTROL, BUREAU OF PUBLIC ROADS

Reported by **J. L. HARRISON, Highway Engineer**

PART I.—EFFICIENT PRODUCTION

FOR many years the construction of concrete pavements has been an important element in the volume of highway work offered contractors. It has also been a large and a growing item in the improvement of city streets and alleys. Probably no figures which purport to give the volume of this work are absolutely accurate, for the very multiplicity of the sources from which such figures must be collected renders absolute accuracy difficult of attainment. However, there is no reason to doubt that recent statistics which show that concrete pavements are now being produced at the rate of about 92,000,000 square yards a year and at a gross annual cost in the neighborhood of $250,000,000, fix the order of importance of this element in the paving contractor's work with sufficient accuracy to justify the statement that it constitutes an extremely large item in the highway construction field.

There can be no doubt that any construction field in which this enormous sum is spent annually ought to be the subject of constant study. This is true without regard to what the field may be, for research and investigations are likely to prove quite as profitable here as they have in such fields as the physical sciences, and in such industries as the manufacture of automobiles. Highways of all sorts are now all but exclusively financed by, and constructed under the direction of governmental agencies with the result that in this field there is a direct relation between efficiency and the tax burden which makes it particularly important that constant effort be made through research and investigation to develop efficiency.

Efficiency is a broad term. Full efficiency in the highway field would begin at least as early as the selection of the routes to be improved. It suggests the development of the principles governing location. It dictates that the designing engineer shall consider the effect of his plans and his requirements on the cost of construction as well as on the probable adequacy of the finished structure. It is as insistent on the proper performance of all necessary operations as it is on the elimination of all that are unnecessary. It urges the contractor toward greater thought in planning and arranging his work and finally it imposes on those in charge of maintenance the requirement that whatever is built shall be so cared for as to yield the greatest service to the community.

Efficiency in the construction of highways is, then, only a part of the greater field of efficiency in highway development and maintenance, and efficiency in the construction of concrete surfaces is but a part of the whole highway construction field. Still, the savings which would result from full efficiency in this field are so large that they justify the annual expenditure of considerable sums of money in research and investigation if thereby approach can be had to complete efficiency. The current difference between ordinary practice and full efficiency will be appreciated when it is known that the studies made by the Bureau of Public Roads indicate that there are very few projects on which the average daily output can not be increased 25 per cent, that on many it can be increased from 50 to 100 per cent, and that as this can be accompanied by some reduction in the amount both of labor and equipment, the labor expended per mile of pavement laid, can, in general, be reduced by perhaps 50 per cent, and charges for depreciation of equipment by an even greater amount.

The series of articles, of which this is the first, is based on extended field studies of going projects by the division of control of the Bureau of Public Roads. In these studies it has been the purpose to determine what efficiency contractors at present secure, what causes prevent the securing of full efficiency, the degree to which these causes interfere with full efficiency, and how they can be remedied. Finally, having determined these facts in what has appeared to be a satisfactory manner, the effect on production of eliminating the ascertained causes of low efficiency has been tried on a number of projects with results that have quite satisfactorily confirmed the accuracy of the studies.

> Studies made by the Bureau of Public Roads indicate that there are few concrete paving projects on which the average daily output can not be increased 25 per cent. On many jobs the output can be increased 50 to 100 per cent, and this can be accompanied by some reduction in both the amount of labor and equipment employed.
>
> Full production on a concrete paving job is 48 batches an hour. By whatever amount a contractor fails to reach this standard he is failing to reach full efficiency in production, assuming that the specifications require a one-minute mix.
>
> The mixer is the bottle neck on every concrete paving job. No matter what material supplies are available, what correlated equipment is on the job, what skilled labor is available, nor what other conditions may prevail, be they ever so favorable, the mixer sets a rigid limit on production which can not be honestly avoided.
>
> To reach full production it is imperative that the discharge be opened and the skip started up at the same instant.
>
> There is an average lag of 5 seconds in charging the mixer after the skip reaches the vertical position, and a lag in discharging which averages 2 seconds. The discharge lag partly offsets the charging lag and the difference should be added to the mixing time.
>
> A slow, indifferent, or incompetent mixer operator may easily lose $35 a day on a job with a daily pay roll of $200.
>
> Loss of time due to mixer trouble averages about 5 per cent and runs as high as 10 per cent. A contractor with a daily pay roll of $200 may lose from $2,000 to $4,500 a season from this cause alone. Careful maintenance of the mixer pays.
>
> Long lines of 2-inch water pipe are a "hang-over" from the days of the 3-bag mixer. The high pressure necessary to force water through them bursts hose, splits pipe, opens joints, and springs the valves on the mixer. Leaky valves are probably the outstanding cause of nonuniformity of consistency. Three-inch pipe will carry an adequate water supply at reasonable pressure and reduce water troubles.
>
> Delays due to lack of prepared subgrade and slow delivery of materials due to inadequate truck supply and other causes are inexcusable on a well-managed job.
>
> Arbitrary interference by inspectors and abuse of discretionary clauses in specifications were the cause of considerable loss to the contractor on more than half the jobs studied by the bureau.

FULL PRODUCTION 48 BATCHES AN HOUR

What is full efficiency? In this series of articles it will be assumed that it is measurably secured when a 100 per cent output is obtained from a properly-equipped job with a crew of proper size. There are, then, a number of distinct fields in which efficiency must be attained before full efficiency is secured. To clarify this statement a little, it may be suggested that when a 100 per cent output is obtained with a 150 per cent working force the work is obviously not performed with full efficiency. Similarly, to obtain a perfect output with a working force of correct size but with a plant which represents an investment 50 per cent in excess of reasonable requirements is not to attain full efficiency. Any discussion of efficiency must deal then, if it is to be complete, not only with production but with labor requirements and with equipment as well. But, because production is at once the point of easiest approach to the general problem in hand, and the point at which current practices most clearly show their failure to reach full efficiency, this phase of the general problem was the first studied and will be the first discussed.

At this point it may be well to raise the question, what is full production on a concrete paving job? The answer will be given categorically: 48 batches an hour—1 batch every 75 seconds. By whatever amount a contractor fails to meet this standard he is failing to reach full efficiency in production, assuming, of course, that the specifications require a 1-minute mix. If a 1¼-minute mix is required, one batch should be obtained every 90 seconds, and if a 1½-minute mix is required, one batch should be obtained every 105 seconds. The 1¼-minute mix, then, reduces output to 40 batches an hour, and the 1½-minute mix to slightly over 34 batches an hour. As, for reasons that will later appear, these reductions in output are not accompanied by equivalent reductions either in daily pay roll or in plant investment, any State, county, or municipality using one of these longer mixing periods, should have very conclusive data at its disposal if it is to justify the additional unit cost of pavement laid, which it must expect to pay.

Because of this fact, and because the current tendency, which is governed by recent tests, seems to be to standardize on a 1-minute mix, the discussions which follow will be based on a mixing period of that length. It should not be forgotten, however, by those interested in obtaining the highest standards consistent with reasonable cost, that current practice in testing concrete to determine the proper time of mix, the proper consistency, etc., is much at fault in that it examines the material discharged from the mixer, or as deposited on the subgrade, rather than the material in place and finished. Between the discharge from any really modern mixer and the final belting, the mixed material is poured into a large bucket, dumped upon the subgrade, shoveled, tramped over, etc., by the puddlers, and smoothed or tamped by a modern finishing machine. The subgrade absorbs some of the water content and some evaporates. All of these operations tend to work the concrete more completely, and particularly to distribute and correct the water film with the result that the material which the belt leaves as a completed slab often is by no means the same material which was discharged from the mixer perhaps a half hour before.

OPERATIONS INVOLVED IN PRODUCTION OF A BATCH

Returning now to the production standard defined above—48 batches an hour—and with the fact in mind that this standard is based on a 1-minute mix, the correctness of this standard may reasonably be examined. The operations which must be performed in turning out a batch of concrete are as follows:

1. The skip must be loaded. This can be done while the previous charge is being mixed. Under the somewhat antequated system which requires that the cement be brought to the job in sacks and there piled along the road, the operation of dumping the cement into the skip will sometimes take longer than mixing the batch. Of course this method of handling cement is as obsolete as are all other purely manual methods, but if it chances to be the method in vogue, men who are strong enough and active enough to do this work within the time available should be put onto this work. Ordinarily, however, handling the cement does not prevent charging the skip within the time available for this purpose, generally from 40 to 50 seconds.

For full production the discharge must be opened when the skip is started up

2. The skip must be raised to a vertical position for the purpose of dumping the batch into the drum. On a modern mixer in reasonably good condition this operation takes from 9 to 10 seconds, during which period the material in the skip is discharging into the mixer only during the last 1½ to 2 seconds.

3. The material in the skip is never fully discharged when the skip reaches the vertical position and sets the timer. The better modern mixers sometimes discharge the skip in as little as 3 seconds after it is in vertical position, if the sand and coarse aggregate are dry; but under ordinary working conditions they tend to require from 4 to 5 seconds, running a little over 5 seconds if the aggregate is wet enough to moisten the cement appreciably. A recently developed 6-bag mixer quite generally shows a lag in the skip discharge which runs over 15 seconds—one fourth minute—and an old-fashioned 8-bag mixer, of which a few are still in use, has been found sometimes to take as much as 40 seconds to transfer all of the charge from the hopper into the drum. As there are a number of standard (5-bag) pavers which will discharge the skip in from a little under 4 to a little over 5 seconds and which, under ordinary working conditions, will regularly discharge within 5 seconds after the skip reaches a vertical position, this period is considered standard for the lag in charging as mixers are now designed.

4. The mix must be retained in the drum the full specification period, here assumed to be 60 seconds.

5. Every mixer should have on it some device for timing. A timer which rings a bell at the completion of the mixing period is the one most commonly encountered. When the bell rings the operator must move the discharge lever and the discharge mechanism must open the discharge before material appears in the discharge chute. These operations generate a lag, partly due to the fact that no human being reacts instantly to any ordinary stimulus and partly to the fact that it takes time for the mechanism to work. Two seconds seems to be about the bottom limit for this lag, which is termed the discharge lag. Three seconds is a more common average; and sometimes it runs as high as four seconds. The lag in discharge tends to offset the lag in charging, but can not reasonably be so treated if it is not a uniform lag. Thus a lag which for successive batches is found to be 2.7 seconds, 3.2 seconds, 2.8 seconds, 3 seconds, 3.1 seconds, 3.3 seconds, etc., over long periods might reasonably be treated as a 3-second lag, though the amount by which this lag exceeds 2 seconds is due entirely to a slow reaction on the part of the operator; but a lag reading 1.9 seconds, 3 seconds, 2 seconds, 4.2 seconds, 2 seconds, 2.2 seconds, 5 seconds, 2.1 seconds, 4 seconds, 3.1 seconds, etc., would appropriately be treated as a 2-second lag because the part chargeable to the operator is not uniform.

6. Finally the mix must be discharged. A wet mix can be quite thoroughly discharged from a good mixer in as little as 8 or 9 seconds, but there is no mixer now on the market which will make a complete discharge in less than 10 seconds, if the consistency is such that it meets present requirements as to slump. This is, in a general way, caused by inadequacy of bucket capacity. Concrete having a slump of about 1 inch flows slowly in the chute and piles up in the bucket with the result that as the bucket fills, the discharging concrete finally backs up into the chute, preventing complete discharge within any reasonable period. To complicate matters further, all mixers dribble a good deal. A recently designed 6-bag mixer will often take well over 20 seconds to complete the discharge. Some of the older mixers take quite as long, a few longer. A full batch mixed to proper consistency can, however, be regularly discharged from most of the standard (5-bag) pavers in 10 seconds if the charge in the drum is "built up" slightly, that is, if a part of the first batch is left in the mixer and the mixer is allowed to run that way. The quantity required is small—just enough to force a full rate of discharge during the whole of the discharge period. As, under full efficiency, a new batch is handled every 75 seconds, there is no practical disadvantage in this as the quantity of material appearing in any batch which could possibly have been in the mixer over 15 minutes would be too small to be of any importance whatever. The discharge time may, therefore, be kept within 10 seconds even when a mix of proper consistency is being produced.

SIMULTANEOUS CHARGING AND DISCHARGING NECESSARY FOR FULL PRODUCTION

The full-production mixing time with all specification requirements met may then be calculated as follows:

Specification mixing time, seconds	60
Lag in charging, seconds	5
Simultaneous raising of skip and discharging, seconds	10
Total, seconds	75

Stated in this manner there is, however, no recognition of the lags described above. It is therefore preferable to state the cycle as follows:

1. Specification mixing time, seconds	60
2. Lag in charging, seconds	5
3. Less lag in discharge, seconds	2
4. Net lag as affecting timer, seconds	3
Normal setting of timer, seconds	63
5. Lag in discharge, seconds	2
6. Simultaneous raising of skip and discharging, seconds	10
Total mixing cycle, seconds	75

It will be observed that in order to reduce the mixing cycle to 75 seconds the skip must be started up at the same instant the discharge is opened. As a matter of fact this is the only device available to the operator for regulating the length of the mixing cycle. Most mixers dribble their discharge enough so that even an experienced operator would have trouble in regulating the time of discharge by observation of the discharge itself. The time required for raising the skip is, however, outstandingly uniform and if an operator is trained to open the discharge and start the skip at the same time, and to close the discharge as the skip reaches a vertical position, he has at once an adequate discharge time, an accurate measurement of the discharge time, and, if the lags in charging and discharging are correctly provided for, the correct mixing cycle.

Two things may be observed here. The first is that where the mixer is so designed that 10 seconds is not an adequate period for discharge this system is not applicable. There are such mixers but they can not be reasonably considered in setting a standard of production. Such mixers may require an extra 5, 10, or even 20 seconds as their minimum mixing cycle. Mixers are not all alike in their efficiency any more than the operators who use them. But to obtain full production a contractor must have an efficient mixer and the above discussion will, it is hoped, assist him in determining which mixers are efficient.

The second matter to be noted is that if the practices here outlined are followed the skip will be in a charging position and material will be flowing into the drum during from 1½ to 2 seconds of the discharge period. This is of no consequence whatever, as the standard drum speed is about 15 revolutions per minute or 1 revolution every 4 seconds. During this overlapping of the charge and the discharge—never in excess of 2 seconds—the drum makes only half a revolution, so the material is not moved up and over in the drum enough to combine with any of the discharging material or to appear in the discharge chute. If the drum speed is so high that unmixed material is discharged, the contractor should reduce the drum speed.

THE MIXER IS THE BOTTLE NECK OF THE JOB

The mixer is the bottle neck on every concrete paving job. No matter what material supplies are available, what correlated equipment is on the job, what skilled laborers or how many are available, indeed no matter what other conditions prevail, be they ever so favorable, the mixer sets a rigid limit on production which can not be honestly avoided. This is so axiomatic that it naturally follows that the problem of securing full production becomes a problem of determining the factors which are reducing the output of

TABLE 1.—*Stop-watch studies of concrete road construction*

[Study No. 15 June 26, 1925. Weather fair. Time, 1 hour, from 1.36 to 2.36 p. m. Number of batches, 24]

Batch No.	Mixing time			Lost time											
	Charge	Mix	Discharge	Truck shortage	Truck delays from causes other than shortage	Mixer trouble		Water supply trouble	Lack of material caused by—		Lack of prepared subgrade	Setting parting strip	Finishers unable to keep up with mixer	Other persistent delays	Miscellaneous delays
						Mechanical	Operative		Mismanagement	Delayed shipment					
	Min. Sec.	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*
1	0 8.5	1 1.0	0 12				0 10.0								
2	8.5	59.5	12				15.5								
3	8.5	1 0	12	2 34.0											
4	8.5	1 0	12				11.5								
5	8.5	1 0	12				13.0								
6	8.5	1 0	12				11.5								
7	8.5	1 1	12	12 12.0											
8	8.5	1 0	12				17.5								
9	8.5	1 0	12				11.5								
10	8.5	1 0	12	1 11.5											
11	8.5	1 0	12	26.5											
12	8.5	1 0	12				9.0								
13	8.5	1 0	12				6.5								
14	8.5	1 0	12				6.5								
15	8.5	1 0	12	6 35.0											
16	8.5	1 0	12					0 14							
17	8.5	1 0	12				6.0								
18	8.5	1 0	12					17							
19	10.0	1 0	12				7.0								
20	9.5	1 0	12					17							
21	9.0	1 0	12					20							
22	9.5	1 0	12				14.5								
23	9.5	1 0	12	18.0			21.5								
24	9.5	1 0	12	30.0			7.0								
Total	3 30.0	24 1.5	4 48	23 47.0			2 48.5	1 8							
Average	9.0	1	12												
Percentage	6	40	8	39.5			4.5	2							

TABLE 2.—*Stop-watch studies of concrete road construction; accumulation of data for study No. 15*

Date	Number of batches per hour	Efficiency of outfit (column 2 divided by 48)	Mixer time				Lost time											
			Charge	Mix	Discharge	Percentage of total time per batch	Truck shortage	Truck delays from causes other than shortage	Mixer trouble		Water supply trouble	Lack of material caused by—		Lack of prepared subgrade	Setting parting strip	Finishers unable to keep up with mixer	Other persistent delays	Miscellaneous delays
									Mechanical	Operative		Mismanagement	Delayed shipment					
		Per cent	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
June 11	16	33.3	0 9.5	0 59.5	0 10	35.0				15.0						50.0		
June 12	36	75.0	9.5	1 1.0	10	80.5				10.5				9.0				
June 12	31	64.6	9.0	1 0	12	69.5		9.0		16.5				5.0				
June 15	36	75.0	9.5	1 1.0	10	81.0		9.5		6.0	2.0			1.5				
June 15	34	70.8	9.5	1 1.5	10	76.0		5.0		13.0	6.0							
June 19	31	64.6	9.0	58.0	12	67.0		3.0		17.5				5.5		7.0		
June 20	38	79.2	10.0	59.0	10	82.0	10.0	1.0		6.0	1.0							
June 20	38	79.2	10.0	59.0	10	82.5	6.0			8.0	2.0			1.5				
June 20	31	64.6	9.5	59.0	10	67.0	14.0	1.0		9.0		9.0						
June 25	10	20.8	10.0	1 1.0	10	23.0	13.0	21.5	41.5	1.0								
June 26	17	35.4	9.5	1 1.0	10	38.0	28.0	1.0	27.5	1.5	4.0							
June 26	24	50.0	9.0	1 0	12	54.0	39.5			4.5	2.0							
June 30	20	41.7	9.5	1 1.0	10	44.0	49.0	3.0		3.0	1.0							
June 30	23	47.9	9.0	1 1.0	12	51.5	30.5	2.0		6.0	10.0							
Average	27.5	57.3	9	1 0	11	60.8	13.6	4.0	4.9	8.4	2.0	.7		1.6		4.0		

Lost time due to improper mixer cycle=60.8−57.3=3.5 per cent.

the mixer, what their order of importance may be and finally, how they may be eliminated.

These factors have been determined in the bureau's studies by making stop-watch readings on selected projects for an hour morning and afternoon each day over such a period as seemed necessary to obtain reasonably accurate data. Table 1 shows a record of this sort. These records, accumulated over a period of from two weeks to a month, tabulated and averaged, give a fairly complete picture of what the rate of production is and of why it is less than it ought to be. Table 2 gives the accumulation of hourly readings for one of the jobs studied, the readings in line 12 being the averages taken from Table 1. Table 3 gives the final summary for a number of jobs, line 15 being the summary of Table 2. These three tables indicate the manner in which the data have been collected, tabulated, summarized, and averaged for use in determining both for individual projects and for this work generally, to what extent production is below standard and what causes are producing this condition. A fairly typical job among those listed in Table 3 (for greater detail see Table 2) shows as the average of fourteen 1-hour readings:

TABLE 3.—*Accumulated summary of 21 time studies of going concrete paving jobs*

Study No.	Batches per hour	Production efficiency	Mixer time			Lost time (per cent)													
			Charge	Mix	Discharge	Improper mixing cycle	Mixer trouble		Truck shortage	Material shortage	Water supply	Subgrade	Setting parting strip	Finishers	Moving mixer	Setting expansion joints	Dumping cement in mixer	Miscellaneous	Total delays
							Mechanical	Operative											
		Per cent	*Sec.*	*Min. sec.*	*Secs.*														
1	26.9	56.0	14	1 7	13	14.0	7.37	0.60	8.76	0.33	2.59	1.82	0.31	3.66	0.69			4.45	44.58
2	27.7	57.7	11	1 4	16	12.3	5.09	.13	7.77		7.83	3.62	.44					4.84	42.02
3	31.6	65.8	11	1 4	9	9.01	6.53		13.60		1.67			.18				2.56	33.55
4	29.6	61.7	11	1 5	8	7.11	.47	1.44	6.48		4.53	3.80	.20	4.58				9.57	38.18
5	12.9	26.9	13	1 3	21	7.7	8.30	.40	43.30		.06				.77	1.28		9.25	71.06
6	19.8	41.3	5	1 3	17	5.4	.05	.83	4.30	4.07		.02				4.36		39.76	58.79
7	16.8	35.0	11	1 15	21	15.5	2.00	13.00	23.00	1.00	.50	4.00					5.00	1.00	65.00
8	24.7	51.5	12	1 9	11	13.2	.50	1.70	27.40	2.00	2.00				.50			1.20	48.50
9	35.6	74.1	13	1 3	12	12.3	.90	.40		.10	.80				11.40				25.90
10	35.4	62.1	10	1 2	11	12.8	16.00	.25			.42				8.51			.09	38.07
[1] 11	23.4	68.8	11	1 25	13	1.9	.70	1.10	20.40		.40			5.10	1.10		.40	.10	31.20
[1] 12	24.0	70.6	14	1 12	19	1.3	.20	1.10	20.70		.50							5.60	29.40
[2] 13	25.4	63.5	9	1 19	2	1.7	1.70	.60	11.40		1.50	11.10	2.10	6.10				.30	36.50
14	35.3	73.5	9	1 6	12	12.5	.30	3.70	1.50		1.60	.60		2.40				3.90	26.50
15	27.5	57.3	9	1 0	11	3.5	4.90	8.40	13.60	.07	2.00	1.60		4.00				4.00	39.20
16	36.5	76.0	9	1 4	5	2.9	1.30	.40	5.60	.50	1.80	1.90						9.60	24.00
[2] 17	14.1	58.1	9	1 21	10	6.6	.40	16.70	10.70			6.30						.70	41.40
[2] 18	23.6	59.0	13	1 25	21	18.2	13.20					6.50	.40					2.60	40.90
[2] 19	25.7	64.2	14	1 23	10	11.4		.90	14.50			2.30						6.60	35.70
20	44.5	92.7	10	1 2	4	0.6					.30							6.40	7.30
21	40.8	85.0	10	56	6	3.1		1.80			3.00							13.30	15.00
22	30.8	64.6	11	59	6	4.8		4.80			3.00	6.70		.4				25.30	45.00

[1] In studies Nos. 11 and 12 the specifications called for a mixing time of 1½ minutes.

[2] In studies Nos. 13, 17, 18, and 19 the specifications called for a mixing time of 1¼ minutes.

Batches per hour		27.5
Efficiency in production	per cent	57.3
Average charging time	seconds	9.4
Average mixing time	do	60.0
Average discharge time	do	10.5
Mixing cycle	do	79.9
Loss of working day, due to—		
(1) Excess length of mixing cycle	per cent	3.5
(2) Slow operation other than mixer cycle	do	8.4
(3) Mechanical trouble with mixer	do	4.9
(4) Inadequate truck supply	do	13.6
(5) Delays due to handling trucks	do	4.0
(6) Trouble with water supply	do	2.0
(7) Subgrade	do	1.6
(8) Finishers	do	4.0
(9) Miscellaneous	do	0.7
Total lost time	do	42.7

The above summary resulting from a two-weeks study of a going project reveals typical construction conditions. Nearly 17 per cent of the working day in this case was being lost at the mixer. Nearly a fourth of this is chargeable to poor handling of the mixer. It will be observed that the timer on this job was not set to take care of the lag in charging. This is not an uncommon condition. Most contractors indeed and many engineers seem to suppose that the material in the skip is all transferred to the drum while the skip is being raised to a vertical position. That this is not the case can be determined readily by getting well up on the mixer and watching the discharge. As most specifications make it clear that the mixing time is the period between the moment at which the last of a batch enters the drum and the first is discharged, the contractor, in this case, was obtaining at least a 3-second advantage over the specification requirements but in spite of this, his operator was losing 3.5 per cent of the working time because of failure to overlap the charge and the discharge which together, as measured from the bell (this includes the lag in discharge), should not exceed 12 seconds.

It may be observed that while, as this job was being operated, the loss of time chargeable to the long mixer cycle amounted only to 3.5 per cent of the working day, its importance would have been somewhat increased had other losses been eliminated. In other words, the length of the mixer cycle was cutting hourly production from a possible 48 batches an hour to slightly over 45 batches an hour or to about 93 per cent of full production. There is here then, and for that matter, in some of the other losses which will be discussed, a difference between the efficiency of an operation itself and the net effect of any lack of efficiency on the day's output. It is possible to develop a study of efficiency on either basis but the latter has been followed in this case.

Poor operation other than failure properly to overlap the charge and the discharge account for a further loss of time amounting to 8.4 per cent. Under this heading are included such practices as dumping batches before the skip is raised, splitting batches, failure to move the mixer while it is mixing, slow reaction to the bell, etc. With an up-to-date mixer it is never necessary to do any of these things. If they are done the operator either has not been properly taught or is deliberately wasting time, or the power plant is so out of order that an expert repair man should be called onto the job at once. The observations made by the bureau's representatives indicate that the general cause of delays of this sort is improper training of operators. Occasionally one is encountered who is naturally slow or indifferent, but it is more common to find that the operator has seen few jobs except the ones on which he has worked and that he does not appreciate that such practices as those here noted throw away a large amount of valuable time.

On this particular job the combined effect of failure to overlap the charge and the discharge and of such items as dumping at the wrong time, splitting batches, slow reaction to the bell, etc., accounts for almost 12 per cent of the working day. With a pay roll running in the neighborhood of $200 a day and a depreciation account of perhaps half that amount, neither of these being unusual on concrete paving jobs, the direct loss from these causes amounts to about $35 a day, a sum quite sufficient to justify the statement that a "cheap" mixer operator is apt to be about the most expensive man on the job.

MIXER TROUBLE EXPENSIVE

In this case mechanical trouble with the mixer accounts for delays aggregating about 5 per cent of the working time. As mixers wear out such delays tend to increase. They are small when the mixers are new, but even on new mixers the studies show that there generally is a little mechanical trouble. This appears to be due to the rigorous conditions under which mixers operate and to the fact that so many operators are trained on the job, rather than to any outstanding deficiency in design. But as the mixer wears out, mechanical troubles increase, and with their increase, the lost time mounts sharply. Five per cent appears to be about the average of losses due to this cause. In some cases they amount to upward of 10 per cent. Under the assumptions noted above, a 5 per cent loss of time amounts to about $15 a day and a loss of 10 per cent to $30 a day. If the mixer works 150 days during the year, losses accruing as a direct result of using equipment that is in poor condition will be seen to range from over $2,000 to $4,500 a year. The first of these is enough to pay for a new mixer in about three years, the second in less than two years. It is not the purpose here to go into the question of replacing old machines with new. Rather, it is the purpose to point out that where old or worn-out machines are in use this very fact will prevent full prodution. Some phases of the problems arising in connection with repairing and replacing machines will be discussed in another part of this series of articles, but it will be sufficient to remark here that as a matter of general observation, there is not enough attention given to the mechanical condition of the mixer, few contractors appearing to realize the real financial aspect of the constantly recurring small losses of time which develop when a worn-out mixer is being operated.

On the job under discussion, the nondelivery of materials accounted for a loss of over 17.6 per cent of the working time. Part of this—13.6 per cent—is chargeable to inadequate truck supply, the balance to improper handling of the trucks. This is a rather typical situation. As a matter of general observation, contractors are prone to provide inadequate facilities for the delivery of material. In the case in hand, the apparent truck shortage was 13.6 per cent, that is, with the truck supply available only 13.6 per cent of the working day, worth on the basis of ordinary pay roll and depreciation charges about $40, was being thrown away because of an inadequate supply of trucks, but the real situation was worse than this. Had the mixer losses been eliminated as well as the losses in time due to other causes, the amount of pavement laid would have been increased only by the 4 per cent chargeable to improper handling of the trucks, that is, to a little over 60 per cent of full production. In short, while lack of proper transportation was only costing this contractor some $40 a day as the job was being operated, the value of this loss would have been about $120 a day had the mixer been capable of normal production. It is hardly necessary to observe that the only appropriate method of eliminating this loss is to supply the necessary trucks. Speeding up the trucks in use is not a proper solution, unless the speed at which they are operated is low, which is an uncommon condition. On the other hand, the correction of low efficiency in the operation of the trucks—an entirely different matter will often measurably improve the apparent truck supply. The transportation question is one of the most important with which the paving contractor must deal—too important to treat at length at this point. It is therefore reserved for further comment in the next of this series of articles.

SMALL WATER PIPES CAUSE SERIOUS LOSS

On this project the water supply caused a loss of 2 per cent of the average working day. Shifting hose is the most common cause of such losses. Poor water pressure is also a common cause. This may be caused by using too small a pump, by slip in the pump, by too small a pipe line, line losses due to poor pipe connections, or to the use of too much water on other parts of the work. Where long pipe lines are used, contractors often try to improve the supply by putting in a booster pump. This results in high pressures which open joints, split pipe, and burst hose. It is not unusual to find that when the valves on the mixer and hose connections are closed, the pressure at the mixer rises to from 250 to over 400 pounds. Such pressures are destructive and wholly unnecessary. Relief valves should be installed to protect the line and the valves in the mixer, but these are often omitted. Larger pipe should be used. The 2-inch pipe now in common use is a "hang over" from the days when the 3-bag paver was standard equipment. Instead of putting in a 3-inch line and having an adequate water supply with reasonable pressure and a reasonable wear and tear on the pumps, contractors are holding to the 2-inch lines much to their disadvantage, for water trouble has been found to be of common occurrence and a supply inadequate for the mixer and the various curing operations the general rule.

Where contractors have long lines of the now commonly prevailing small sizes of pipe, some relief from the high pressures incident to pumping any considerable volume of water through these lines can be had by cutting the line and instead of using booster pumps, running the delivery from the first pump into a sump from which the second pump will draw its supply. A great deal of the trouble with short water supply is directly chargeable to line leakage and this, in turn, is chargeable in no small degree to high pressure in the line. With pressures commonly exceeding 250 pounds ordinary connections are opened, and all sorts of trouble resulting in leakage occurs. Even the valves in the mixer are sprung and the leakage which occurs here sooner or later becomes an obstacle to the regulation of the water content of the concrete produced. This is a serious matter, since the strength of concrete is adversely affected by even a comparatively small increase in its water content. Moreover, variations in the water content have an adverse effect on the ease and rapidity with which the pavement is finished and make the production of a very smooth surface difficult.

Where there is any considerable leakage the amount of water improperly reaching the interior of the drum varies with the pressure in the line and with the period between batches. As on all but the best-managed jobs both of these are highly variable, as soon as there is any considerable amount of leakage, real uniformity of consistency becomes quite unattainable. So really serious is this matter that it seems proper to suggest that, as mixers are now operated, this is the outstanding cause of nonuniformity of consistency and one of the most important causes of poor surface finish.

There is therefore very good reason for urging contractors both in their own interest and in the interest of the States and municipalities for which their work is done, to install pipe lines of proper size, fitted with proper relief valves in order that thereby an adequate water supply delivered at a reasonable pressure may be insured. There is also reason to suggest that manufacturers install better and stronger valves in their mixers.

Delay due to lag in finishing is not of common occurrence. Finishing machines are reliable mechanisms and commonly give the contractor little trouble. The 4 per cent loss from lag in finishing recorded on this project is unusual. Occasionally a complicated or cumbersome process in finishing is required and in such cases there sometimes is a failure to keep the finishing up with the mixer. Sometimes too dry a mix is required by the inspector. However, the current tendency is toward simplifying the finishing operations so, except where the inspection is too exacting, there is seldom any difficulty encountered here. Incidentially it may be remarked that the essence of rapid finishing is uniformity in consistency and a consistency of proper slump. Well-mixed concrete of the proper consistency—about 1-inch slump—put down by a modern sliding finisher may be depended upon to be of such uniform smoothness that one man with a long-handled float can finish any output obtainable from a standard mixer. This, of course, implies that the concrete shall be of uniform consistency as well as that it shall be of proper consistency and that the finishing machine shall be correctly operated. In short, there should be no losses of time due to finishing and if any are developing there is something wrong which should be easily corrected.

Occasionally an inspector in his desire to obtain the driest mix possible will attempt to reduce the slump to ½ inch or less. Whatever the theoretical result of such a practice may be, in increasing the strength of the concrete, the practical result under field conditions is to render the concrete extremely hard to work. With a slump as low as this, the finishing machine can not keep up with the mixer. Moreover, in at least two cases which have come to the attention of the writer, the finishing operation did not satisfactorily compact the bottom third of the pavement. Under field conditions there would, therefore, seem to be valid reason for maintaining a somewhat wetter mix than laboratory tests would indicate to be the most desirable. There is also to be considered the fact that the subgrade absorbs moisture from the concrete and that some evaporates, as a result of which processes, the concrete as finally left by the finishing machine, particularly in hot, dry weather, often is visibly drier than when it left the mixer. If satisfactory production is to be secured, the finishing operation can not be unduly slowed down. There is, here, then, room for a good deal of practical study, for although it is important that the best possible concrete be produced, it is also desirable that this be done on the basis of carefully collected data rather than on the basis of personal opinion, and that theoretical considerations be not allowed to interfere with production unless it can be clearly shown that the results to be obtained can and do justify this interference.

DELAYS DUE TO LACK OF SUBGRADE AND OTHER CAUSES

As with the finishing so also with the subgrade, if the crew is not doing its work rapidly enough to keep the subgrade in proper condition and so far in advance of the mixer as to entirely avoid delays from this cause, either the foreman in charge needs instruction, additional men or equipment, or the job needs a new foreman. There should be no excuse whatever for letting the preparation of the subgrade interfere with production at the mixer.

Miscellaneous delays are of small consequence on this job. However, the list of them as affecting production a little here and a little there, not often but occasionally during a long season, is long. Special work at bridge structures falls in this category. Often specifications require a special treatment of the pavement at such points. This special treatment causes delay, generally only because it does not occur often enough so that the crew becomes familiar with the operations involved. Any new operation takes time because men not familiar with it hesitate to proceed lest a mistake be made. Most men working on a construction job do their tasks automatically. Many, if not most, feel confused if any judgment is required. No new or unusual task, no matter how simple, can be handled automatically and most such tasks for a time require at least a little judgment even on the part of the common laborers. For that reason, whatever is new or unusual is done slowly and output suffers until the operations involved are so well learned through repeated performance that they become automatic.

"Waiting for reinforcing rods" is a minor delay typical of a whole series which may be classed under the general heading of "delays due to the failure to deliver minor materials." Generally such delays are due to the failure of some foreman to keep track of the stock on the job but, if reinforcing material, parting strip, prepared joints, etc., are required, the lack of such materials at the site of the work is quite as effective in stopping production as the failure to deliver cement or sand or gravel. Laying a concrete pavement really is a manufacturing process. Modern manufacturing methods do not depend for their success on large stocks. They depend, rather, on the maintenance of uniform balanced rates of delivery. This is as true of a concrete paving operation as it is of an automobile plant. Materials are not stocked. They are ordered for delivery as needed and such stocks as are maintained are like the balancing tanks in a water supply—they absorb the delivery when it exceeds the rate of use and they augment the delivery when it falls below the rate of use. In the case of minor materials perhaps this is not as obvious as in the case of the materials of major importance, but the fact is that for all materials the rate of delivery at the mixer must harmonize with the rate of use at the mixer and any failure to attain such a treatment of the material supply, whether major or minor, is too disastrous to pass unnoticed.

There also is a whole series of minor causes of delay not so easy to cover in a general statement, such as: Broken batch meter; getting the mixer under low electric wires; failure to dump all the batch into the skip; over-wet batches thrown away; water pipes frozen; and forms knocked down by trucks. In a general way, delays of this sort are due to accidents or to accidental conditions and as their causation can not be foreseen, they can not well be avoided. However, they are not a serious factor on most jobs but are mentioned because they illustrate the innumerable minor matters which interfere with securing full production.

THE DAMAGE DONE BY INEXPERIENCED INSPECTORS

Finally no discussion of the causes affecting production would be complete without reference to the inspector. He is a necessary, though, it must be regretfully recorded, not always a favorable factor in the production problem. Generally he is a well-intentioned man who knows but little about the broader technical aspects of the work he is inspecting, very desirous of protecting the interests of those he represents, but not properly equipped to pass on the numerous technical questions that arise on any concrete paving job. He occupies a difficult position because, after all, the contractor's primary desire is production, whereas the State's primary desire is quality. The two are seldom in real conflict, for the best organized and most productive jobs generally do the best work; but high production requires a quick answer to any problem that arises and this, in turn, requires a broad knowledge of technical values or, in the effort to protect the interests of his employer, the inspector will find that he is needlessly injuring the contractor. Without this broad technical background, the inspector is apt to be further handicapped by the fact that he has little appreciation of the real nature of a going construction operation. Here he sees a few men, there a few more. At other points he knows that others are working. He seldom realizes that what he is observing is a machine, the parts of which are so synchronized that to stop one part in effect causes all of the parts to stop functioning. Yet this is literally true.

The modern concrete paving organization, when well developed, is as perfect a machine as a well-built automobile. The subgrade gang is balanced to produce each day the subgrade that the mixer will use the next. The form-setting gang is balanced against the mixer performance in the same way. At the batcher plant the force is kept in harmony with the mixer's utilization of material. The office keeps materials flowing in as they will be needed. In such a balanced production scheme—and this is the very essence of efficiency in production—the injection of any order that stops production at one point stops the whole machine as effectively as taking off a wheel or pulling out the engine stops an automobile. To stop one gang is to stop the whole job. Worse than that, the time lost directly may be of less importance than the time lost indirectly. The writer watched one afternoon an irresponsible inspector while he tinkered with the water gauge on the mixer for a half hour in a misdirected effort to further reduce the slump of concrete which was already well within the specified requirements. The result was a series of batches so dry that the contractor had to stop the mixer for half an hour in order to finish them. This half hour's delay was important enough in itself, but it was a couple of hours before the contractor was able to get his organization to functioning again with its accustomed smoothness.

Although arbitrary acts on the part of inspectors are serious enough, a larger trouble lies in the constant effort of many inspectors to inject new ideas into work on which they are engaged. Some one desires the subgrade prepared in a new way. The finish, it is thought, might be improved. The concrete might be mixed a little drier. It would be improved if mixed a little longer. There are dozens of such requests. It matters not at all that the work now meets the specifications which are in force. If some one thinks that the product can be improved, the contractor will be urged, even forced, into making the desired change. He is told that the change will cost him nothing—may save him money—and will get better results. The point overlooked is that changing the practices on a going job requires more than an order. Men must be trained to perform the new duties properly, and the job may have to be rebalanced to meet the new methods. Often the contractor does not know how to train the men as he has never seen the new ideas in use. He, therefore, hesitates because he knows that such changes may involve large intangible losses, clear enough to him but hard to explain to an inspector.

Of course, the time to decide on what methods and practices are to control on a construction job is when the specifications are drawn. Then, when the job is started, the interpretation which will be placed on new clauses or on any of the older clauses that may be in doubt should be made clear, and the practices that are to be required should be developed as the organization is brought into production. Thereafter all connected with the job should appreciate that while the contractor may request premission to use new methods the inspector and, for that matter, the State, has no right to modify the requirements made on him, unless infraction of specific provisions or commonly accepted interpretations of the specifications can be shown.

ABUSE OF DISCRETIONARY CLAUSES IN SPECIFICATIONS

The clause, "or to the satisfaction of the engineer," and its various equivalents is often written into specifications for the purpose of giving the engineers broad discretionary powers in the interpretation of clauses governing methods, tolerances, etc.; and if a contractor objects to an order or to a request, some clause of this sort is called to his attention. For this reason it seems advisable to remark in this connection that as a general legal principle clauses of this sort give engineers no authority to modify the plain intent of specifications defining methods, tolerances, etc., and that contractors are under no obligation to be governed by orders which make these clauses more than customarily rigorous or which increase the cost of the work. As an illustration, if the specifications clearly state that "the slump shall not exceed 1 inch," a clause of this nature can not be legally invoked to require a contractor to produce concrete having a slump of not more than half an inch. Similarly, if the specifications provide that "irregularity in the finished surface shall not exceed one-fourth inch when measured under a 10-foot straightedge," a clause of this character can not be lawfully invoked to require a contractor to remove areas that are only one-eighth inch high. If the contractor desires to use other than specified methods, etc., and the specifications are not to be followed, then the discretionary clause gives to the engineer broad authority to require that whatever is done under these new methods shall be done "to the satisfaction of the engineer." But in no event are such clauses blanket authorizations for the engineer to make such requirements as he may please.

If it were possible to bring inspectors generally to a realization of the fact that their sole duty is to see that the contractor works within the specified limits, that they are not present to "improve the quality of the work," or to make experiments at the expense of the contractor, the effect on production would be gratifying to many contractors. In making stop-watch studies on going projects it has seldom been possible to separate the effect of improper inspection from other causes which perhaps also contributed to slow operation. Sometimes the fact that an inspector's order is responsible for a procedure that is causing delay has not developed until a good deal of the record has been taken. A case in point was a job where the records showed slow operation of the mixer. In general, the truck supply was adequate and the work well organized, but the mixer operator would not raise his skip until the discharge had been completed. This of course materially reduced his daily output. After the full record for this job was taken it was learned in a chance conversation with the inspector that he would not permit the contractor to produce over 40 batches an hour. There was no specification warranting such a limit on production. The inspector gave no reason for having imposed it. He merely asserted that he thought that production enough. Conditions of this kind as well as the possibility that there might be some difference of opinion as to what is proper inspection, have prevented any general effort to separate the time lost through poor inspection, but some idea of the importance of this element may be gained by the general statement that poor inspection has caused losses of some consequence on more than half the jobs on which the bureau's representatives have made studies during the current season. It is not, therefore, a factor which can be ignored in a study of the causes influencing underproduction.

In presenting these aspects of the inspection problem, the effect of improper inspection and of improper policies governing inspection have been dealt with at some length. It is, therefore, only fair to remark that it is not the purpose to advocate careless or indifferent inspection. Rather the quality of inspection should be improved. The point is that a legal and a moral prerequisite is correct specification writing. If a State desires its concrete pavements finished to a maximum variation of one-eighth inch in 10 feet this can be done. It is perfectly proper that it require such accuracy of final finish and it is practically possible to obtain it. But it is not legally or morally proper to require such a finish under clauses allowing a one-fourth inch variation. Refinements in methods and the reduction of tolerances increase cost. Inspection should be rigid and good work should be insisted on, but back of this there should be a clear specification setting forth what is desired so that the contractor may have a fair opportunity to estimate accordingly.

SOME IMPORTANT GENERALIZATIONS

The above covers the more important causes of low production. It would be possible with the data in hand to strike averages and to say that this or that is the most important factor in causing low production and to ascribe to it a percentage relation to the whole series of causes, but little could be gained by so doing, for each job is a separate problem and general averages have little relation to the specific conditions there prevailing. Studies such as those the bureau has conducted in this field do, it is true, warrant some generalizations. Mixer operators are, for instance, quite generally undertrained, with the result that few mixers show a proper mixing cycle. Transportation generally is inadequate. Often it is woefully so. As a matter of observation it is apt to be poorly operated. The water supply also is generally inadequate; while the mixer is usually pretty well supplied, the quantity available for curing is commonly insufficient. This list could be extended but to do so has little value for the contractor whose problem is to improve production on his own job of necessity must realize that generalities and averages are a poor approach to the solution of his specific problems. It is well enough that he should know that mixers are seldom properly operated and that the truck supply is commonly inadequate, but before he can set about the correction of his own low production, he must have the facts as to specific conditions on his own job. These he should obtain just as they have been obtained for these studies by making stop-watch records over such periods as may be needed in order to clearly show what delays are occurring. With the fact clearly in mind that his job ought to be producing 48 batches every hour, it is as simple a problem to determine what delays reduce the output to 35 batches as it is to measure the top of a desk with a 2-foot rule. For studies of this sort a stop watch is desirable, though they can be made with fair accuracy from the second hand of an ordinary watch. If contractors in this field would equip their timekeepers with stop watches and would require them to take an hour's reading morning and afternoon, setting down the results as in Table 1, and summarizing them as in Table 2, the information obtained would be invaluable.

Inspectors and superintendents should also be equipped with stop watches. This really needs no argument in its support. No inspector would be allowed to pass judgment on the dimensions of a bridge if the best rule he had was graduated only in feet. Neither would a superintendent be allowed to build a bridge with such a rule as his guide. Yet hundreds of inspectors and hundreds of superintendents constantly direct work to-day where seconds are valuable on the best guesses they can make from the second hand of an ordinary watch. It may be interesting to add that in the fastest job the bureau's representatives have studied—the only job so far encountered on which the output was consistently maintained at above 90 per cent of full production—the superintendent had and frequently used a first-class stop watch.

(Continued from page 193)

decreases from 15° to 45° but at 60° is slightly greater than at 15°. It is supposed that the stresses in the right-arch sections will take the shortest course to the abutments, which is at right angles. The stresses in the triangular sections *A E B* and *D C F* will be transmitted with greater intensity to the obtuse angle of the arch. The result should be the greatest intensity of stress at the obtuse angle.

On the contrary, suppose arches had been tested which included no right section as at *X Y Z W*, Figure 12. It is possible in this case that tension might be shown to exist at the acute angle. This did not occur (except in negligible amounts) in the arches tested. For this reason it is evident that these tests include only a special condition of width of arch in relation to skew, and it is not possible to develop reliable mathematical formulæ from the existing data.

WIND RESISTANCE OF MOTOR VEHICLES

A PROGRESS REPORT OF TESTS CONDUCTED BY THE KANSAS STATE AGRICULTURAL COLLEGE IN COOPERATION WITH THE BEREAU OF PUBLIC ROADS

By L. E. CONRAD, Professor of Civil Engineering, Kansas State Agricultural College

AS A result of tests conducted during the past four years by the engineering experiment station of the Kansas State Agricultural College with the financial assistance of the Bureau of Public Roads, the resistance of wind at various velocities to a number of standard makes of motor vehicles has been measured and the relation between the wind resistance and relative velocity for these makes has been tentatively established. Both the total wind resistance and the resistance per square foot of the projected area of the vehicle have been determined for each of the makes tested; and from the results of the tests the horsepower required to overcome the wind resistance to each type of car has been computed.

The equations representing the relation between wind resistance and velocity as derived from the experimental data for each car are all of the parabolic form expressed by the type $P = CV^a$, in which P is the wind pressure, V the wind velocity, C a coeficient determined experimentally for each car, and a is an exponent also determined experimentally for each car.

The form of the equation is the same for the total and unit resistance, the only difference being in the coefficient, which for the unit resistance is obtained by dividing the coefficient for total resistance by the projected area of the car.

The average value of the exponent for the 14 stock cars tested is found to be 2.14. In no case is the value below two and in only one of the equations does it equal two.

Of the cars tested a Buick touring car was found to have the highest wind resistance; a Ford roadster the lowest. The resistance of the former was approximately 50 per cent higher than that of the latter. The other makes and styles of body tested have resistances between these two extremes, but the variation does not appear to be consistently related to either the make or the body style. The resistance of open cars does not appear to be consistently higher than that of the closed models as some experimenters have found.

The average equation for the unit resistance of the cars tested is found to be $P = 0.00149\ V^{2.14}$. The same relation for flat surfaces is generally considered to be $P = 0.003\ V^2$, and other experimenters with automobiles, assuming an exponent of 2, have arrived at coefficients ranging from 0.00234 to 0.00340. If for purposes of comparison, a definite velocity, say 35 miles an hour, be substituted in the average equation above the relation is found at this speed to be equivalent to $P = 0.00250\ V^2$.

RESISTANCE OF FULL-SIZED CARS MEASURED IN WIND TUNNEL

After a year's work with natural winds which proved to be extremely variable and gusty it was found to be necessary to resort to the construction of a wind tunnel in order to produce by artificial means a reasonably constant wind velocity. The wind tunnel constructed (see fig. 1) is of rectangular cross section, 10 feet high by 12 feet wide at the throat and 50 feet long, these dimensions being increased at the test section to give an additional 27.5 square feet, which is about the average projected area of all cars measured to date.

The automobiles are set on a swinging platform, forming part of the floor, and supported by two wooden beams which are suspended from the roof by chains outside the tunnel. Side sway is prevented by four ball-bearing rollers running between two longitudinal steel tracks. When a car is blocked on this platform and the fan starts drawing air through the tunnel, the horizontal force exerted on the car causes the platform to swing backward. This horizontal force is measured by a spring balance which is connected to the front end of the platform under the floor, and which is tightened until the platform has returned to its original or zero position.

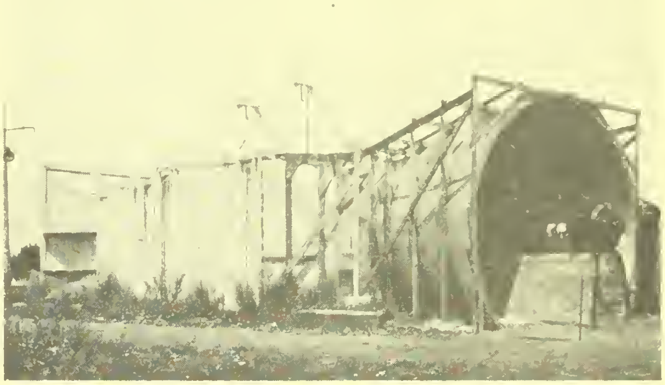


FIG. 1.—The tunnel used in the wind-resistance tests

The horizontal force or pull is made up of two factors, the wind resistance, or drag, of the car at the given air velocity, and the resistance of the platform. With no car in the tunnel, the platform resistance is too small to be measured. It was thought, however, that the presence of the car above the platform probably would increase the resistance of the latter. Accordingly, the car was suspended from the roof of the tunnel so that the wheels almost touched the platform. The resistance of the platform in this case was found to be quite appreciable; and this resistance was subtracted from the total pull in all tests. Figure 2 shows this correction, which is called the platform correction, and which amounts to a little over 3 pounds at 40 miles per hour.

The advisability of installing an air straightener or honeycomb at the entrance of the tunnel was investigated. The installation of a honeycomb usually decreases the maximum obtainable air velocity from 15 to 25 per cent. Furthermore, the honeycomb would have to be removed between tests to allow for changing cars. In view of these facts, it was decided not to use a honeycomb unless the condition of the air stream was found to be extremely bad. The first test was made by hanging streamers at various points in a cross section 7 feet ahead of the car, and noting the direction taken by each streamer. The direction in every case was parallel to the axis of the tunnel.

This test proving satisfactory, a second test was made to determine the air velocity at various points by dividing the cross section of the tunnel into 25 equal rectangular areas and determining the velocity at the center of each area. The results of 12 series of these tests gave an average variation of 6 per cent from the mean velocity, which was not considered sufficient to warrant the installation of a honeycomb. The air speed was measured by a pitot tube and a differential manometer sensitive enough to measure differences in presssure of 1/100 mm. of alcohol.

As remarked above, the cross-sectional area of the tunnel at the test section was increased by 27.5 square feet to compensate for the area of the car. While this added area is almost exactly equal to the average area of all cars tested to date, the area of some of the stock cars departed from the average by 4 or 5 square feet, while the area of one racing car differed by 13½ square feet. In computing the test results, therefore, the velocities for each car were corrected for this difference in area.

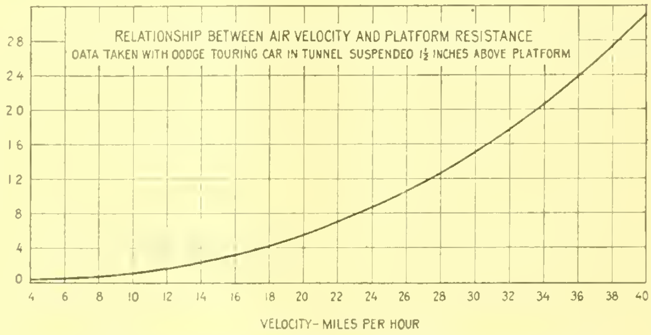


FIG. 2.—Curve used to make correction for wind resistance of platform

EFFECT OF SIZE OF TUNNEL AND OTHER TEST CONDITIONS

In order to ascertain whether the tunnel was enough larger than the automobiles to make the results comparable to actual road conditions, it was decided to determine the velocity of the air currents at certain significant points around the car both on the road and in the tunnel. One series of these tests has been run in which a very close correspondence was found between the action of the air currents around the car in the tunnel and on the road.

The effect of natural winds on the test was found to be quite pronounced; both the air speed and pull indicator becoming quite erratic. Consequently no tests have been run with a natural wind in excess of 2 miles per hour.

To determine the effect of rotation of the wheels a Dodge touring car was raised above the tunnel platform by four stilts. The front and rear hub caps on each side were connected together by leather belts, and idler pulleys were installed to prevent the belts from rubbing on the running boards. The differential of the car was locked so that both rear wheels would run at the same speed, and, after calibrating the speedometer, aligning the front wheels, and tightening the belts, it was possible to run all four wheels at any desired speed. To eliminate the effect of the stilts, belts, and driver a series of tests was run with the wheels stationary. After warming up the motor another series of tests was run with the wheels rotating at speeds corresponding to the various velocities of the air in the tunnel. That there was no appreciable difference between the results of the two series of tests was probably due to the shielding effect of the fenders on the car tested.

The effect of the automobile fan when the engine was running still remained to be evaluated. This was determined by setting the shift lever in neutral and running the automobile engine alone. It was impossible to obtain any movement of the platform indicator even by racing the engine, which is a logical result when it is considered that most of the air after passing through the automobile fan strikes some portion of the engine or car and gives up most of its energy before leaving the car.

COMPLETE TESTS ON 15 CARS

In view of the foregoing results, it appeared that the wind tunnel could be relied upon to furnish reasonably accurate values of the wind resistance, and complete tests were run on 15 different cars. So far as possible the sedan, touring, coupé, and roadster models of several different brands of cars were tested. The tests were made in the following manner. At a time when there was little or no natural wind the automobile to be tested was backed into the wind tunnel, centered on the swinging platform, and securely fastened so that it could not be drawn back into the fan. The zero position of the platform was next determined. The millimeter scale of the manometer was adjusted so that its zero corresponded to the zero position of the meniscus in the tube, and the arrangements for the test were completed by connecting the pitot tube to the manometer.

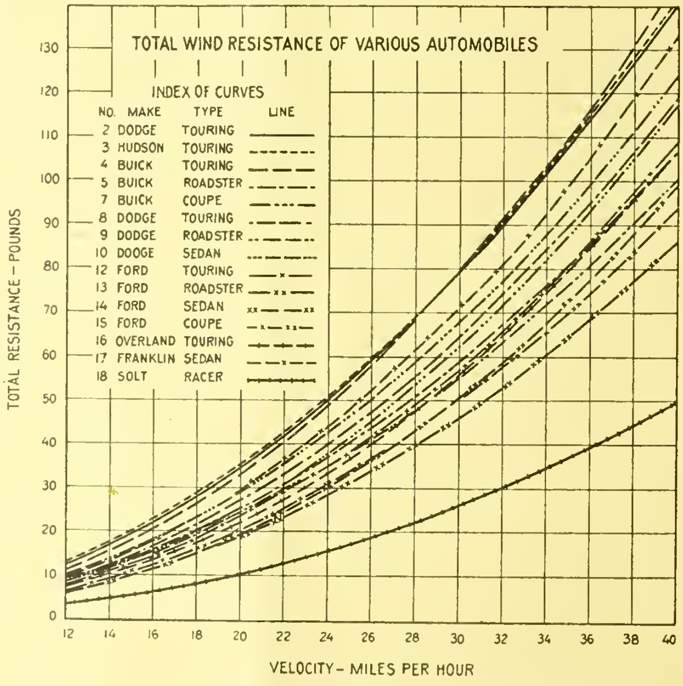


FIG. 3 —Curves showing total wind resistance to automobiles at various wind velocities

The fan was started at its lowest speed; and immediately the platform swung toward the fan. The platform was then returned to its original position by tightening the spring balance adjustment; and when the pointer indicated that the platform was in its original position, the manometer and spring balance were

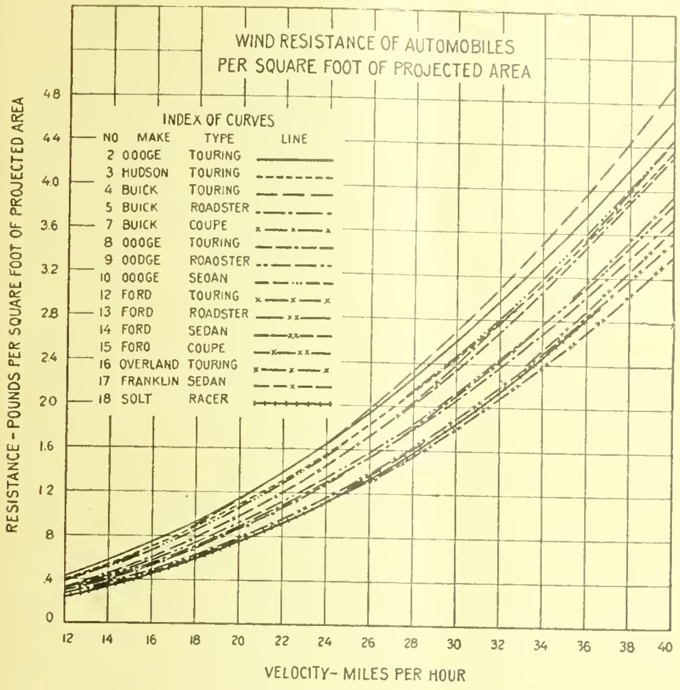


FIG. 4.—Curves showing unit wind resistance to automobiles at various wind velocities

read simultaneously. The process was then repeated for seven different air velocities, obtained by short circuiting out a portion of the armature resistance, and these data formed one test. At least two and usually three tests were run on each of the 15 cars.

The air velocity was recorded in terms of centimeters of alcohol and reduced to miles per hour in the usual manner. All velocities were reduced to an assumed standard atmosphere of 70° F. and 29 inches of mercury which represents a year-round average at Manhattan, Kans.

The spring balance was read in millimeters and reduced to pounds from a calibration chart, a correction being applied for any slight departure of the platform pointer from the zero position at the time the manometer was read. The resulting figure was multiplied by an air density factor to obtain the pull in terms of standard air, and from this value the platform resistance was subtracted, the result being the resistance of the automobile at the corresponding air speed.

The projected area of the car was next determined by tracing the outline of an enlarged photograph with a planimeter. All these photographs were taken at a distance of 200 feet, using a long-focus lens. The photograph thus obtained was enlarged about 10 times which magnified the image of the car 100 times. The relation between the size of the photograph and the size of the car was obtained from a scale which was photographed on the car.

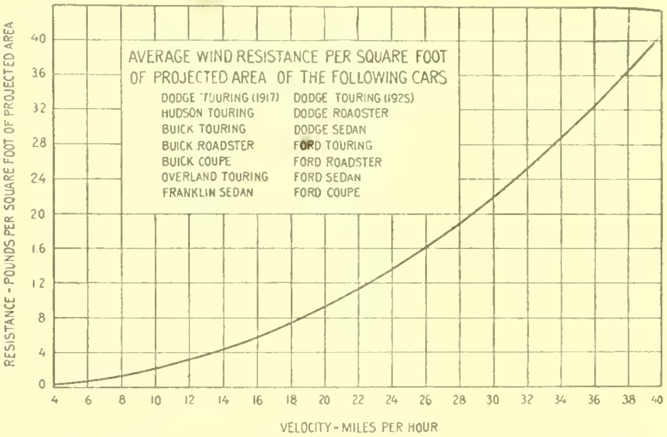


FIG. 5.—Curve of average wind resistance at various velocities for 14 stock cars

THE TEST RESULTS SUMMARIZED

The results of the tests run are summarized in Table 1, and are shown graphically by the curves of Figures 3 to 6. The equation representing the experimental data for each car was derived by the method of averages. These equations are all of the parabolic form and the exponents are not far from two, which has long been assumed to be the correct power of an equa-

TABLE 1.—*Summary of results of tests of wind resistance of automobiles*

Vehicle No.	Make of vehicle	Type of body	Model	Windshield	Top	Curtains	Projected area	Equations[1]		Protuberances, remarks, etc.
								Total pull on car	Pull per square foot of projected area	
							Sq. ft.			
2	Dodge	Touring	1917	Shut	Up	Off	30.52	$P=.0806\,V^{2.03}$	$P=.00264\,V^{2.03}$	No extras; no spare tire.
3	Hudson	do	1918	do	do	do	32.56	$P=.0880\,V^{2.00}$	$P=.00270\,V^{2.00}$	Motometer, vizor, bumpers, spot light, tool box on right running board, two spare tires on left running board.
4	Buick	do	1923	do	do	do	29.48	$P=.0550\,V^{2.14}$	$P=.00186\,V^{2.14}$	Motometer, vizor, bumpers, spare tire at rear.
5	do	Roadster	1923	do	do	do	28.42	$P=.0512\,V^{2.07}$	$P=.00180\,V^{2.07}$	Motometer, spare rim at rear.
7	do	Coupe	1922	do			29.48	$P=.0346\,V^{2.20}$	$P=.00117\,V^{2.20}$	Motometer, vizor, bumper at front, spare tire at rear.
8	Dodge	Touring	1925	do	Up	Off	27.44	$P=.0450\,V^{2.14}$	$P=.00164\,V^{2.14}$	Motometer, bumpers, balloon tires, spare tire at rear.
9	do	Roadster	1925	do	do	do	25.90	$P=.0478\,V^{2.07}$	$P=.00185\,V^{2.07}$	Motometer, bumpers, balloon tires, spare rim at rear.
10	do	Sedan	1924	do			27.75	$P=.0585\,V^{2.08}$	$P=.00211\,V^{2.08}$	Motometer, vizor, bumpers, balloon tires, spare tire at rear.
12	Ford	Touring	1925	do	Up	Off	27.49	$P=.0326\,V^{2.20}$	$P=.00118\,V^{2.20}$	Balloon tires, spare tire at rear.
13	do	Roadster	1923	do	do	do	25.60	$P=.0275\,V^{2.18}$	$P=.00108\,V^{2.18}$	Spare tire at rear.
14	do	Sedan	1924	do			27.18	$P=.0191\,V^{2.32}$	$P=.00070\,V^{2.32}$	Motometer, vizor, bumpers, balloon tires, spare tire at rear, folded luggage carrier on left running board.
15	do	Coupe	1925	do			26.55	$P=.0289\,V^{2.19}$	$P=.00109\,V^{2.19}$	Vizor, balloon tires, spare tire at rear.
16	Overland	Touring	1923	do	Up	Off	27.04	$P=.0302\,V^{2.22}$	$P=.00112\,V^{2.22}$	Spare tire at rear.
17	Franklin	Sedan	1922	do			29.70	$P=.0422\,V^{2.18}$	$P=.00142\,V^{2.18}$	Bumpers, spare tire at rear, folded luggage carrier on left running board.
18	Solt	Racing	1923	None.	None.	None.	14.05	$P=.0126\,V^{2.25}$	$P=.00089\,V^{2.25}$	Motometer, 2 spot lights, no head lights, spare wire wheel on right side.

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[1] P=Pull in pounds. V=Air velocity in miles per hour.

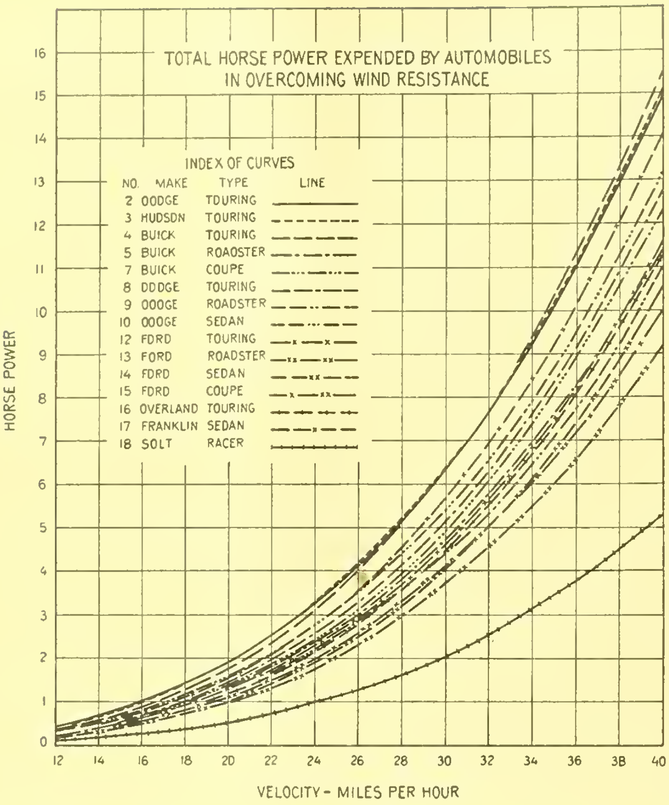

FIG. 6.—Horsepower required to overcome wind resistance to automobiles at various wind velocities

tion representing wind pressure. In one of the equations the exponent is exactly two, and in no case does it fall below that figure. The average value for the 14 stock cars tested is 2.14.

Figure 3 shows the curves for the total resistances of 15 different cars. The lowest one, No. 18, represents the resistance of a specially made racer without top, windshield, or fenders. The other 14 are stock cars which were borrowed for the tests from automobile dealers and private owners.

Figure 4 shows the curves for the unit resistances of the same cars. These were obtained by dividing the total resistance of each car by the projected area of the car in square feet. The highest of these curves, representing a Buick touring car, is approximately 50 per cent higher than the lowest, which represents a Ford roadster. In other words, the extremes vary only about 25 per cent from the average. This average curve is shown in Figure 5. The variation in unit resistance, apparently, does not accord with the make of car or type of body. The open cars are not consistently higher than the closed, as some experimenters have found.

The equation of the average curve (Fig. 5) is $P = 0.00149\ V^{2.14}$ where P is the pressure per unit area. In order to compare this with the values derived by other experimenters who assume the exponent to be 2, some definite velocity must be assumed. At 35 miles per hour, the equation would read $P = 0.00250\ V^2$. This is somewhat less than the pressure on a flat surface which is generally considered to be $P = 0.003\ V^2$. Experimenters heretofore have arrived at values of the coefficient for automobiles ranging from 0.00234 to 0.00340.

The horsepower required to overcome the air resistance of the cars tested is shown by the curves of Figure 6. These were calculated from the curves for total resistance by using the formula $H = \frac{PV}{375}$, where H = horsepower, P pressure in pounds, and V = velocity in miles per hour.

METHODS OF CURING CONCRETE IN A SEMIARID CLIMATE STUDIED

Studies of Methods of Curing Concrete in a Semiarid Climate, by Harrison F. Gonnerman and C. L. McKesson, has just been published as Bulletin 15 of the Structural Materials Research Laboratory, Lewis Institute, Chicago.

This investigation was conducted at Sacramento, Calif., as a cooperative research by the California Highway Commission and the Structural Materials Research Laboratory. The climatic conditions were quite unfavorable for the proper curing of unprotected concrete, but are typical of those encountered in semiarid regions.

The experiments were carried out on 7 by 10 by 38 inch Portland cement concrete beams made out of doors and cured in the open. Curing methods included:

1. Covering with wet earth.
2. Covering with asphaltic paper.
3. Surface application of flake calcium chloride.
4. Surface application of sodium silicate.
5. Air exposure.

Five hundred and eighteen beams were tested in cross-breaking with the cured surface in tension at ages of 3 to 90 days. The hardness of the cured surfaces of the beams was measured by a ball-indentation test. Compression tests were made on 175, 6 by 12 inch cylinders and prisms, in order to secure a measure of the quality of the concrete in a standard test.

In comparing the relative efficiency of the different curing methods, the strength of the concrete cured with earth wet 7 and 14 days (which showed practically identical results) was taken as the standard. The strength ratios for the different methods of curing were based on the following values of modulus of rupture of 1:.2 2:3.0 concrete: 7 days, 445; 14 days, 470; 28 days, 535; 90 days, 600 pounds per square inch. (The 28-day compressive strength was 4,000 pounds per square inch.)

The principal conclusions from the tests are—

1. A curing method is efficient which maintains the moisture content of the concrete during the early stages of hydration, about equal to the original quantity of mixing water. Wet-earth curing gave the best results; this method apparently supplied moisture in sufficient quantity to replace losses due to evaporation and to absorption by the subgrade. Curing methods which permitted high evaporation losses gave concrete of low strength.

2. The tests showed that concrete cured under earth wet for 7 or 14 days was only slightly stronger than when cured under earth wet for 3 days. In view of this and of the small rate of increase in strength after the fourteenth day it may be concluded that with temperature no lower than prevailed in these tests (70° F.) concrete pavements cured by covering with wet burlap

(Continued on page 212)

HIGHWAY RESEARCH PROJECTS TABULATED

AT THE request of the Bureau of Public Roads the highway research board of the National Research Council has recently completed a partial survey of research projects recently completed or now in progress under the auspices of State highway departments, universities and State experiment stations. The bureau desired information especially with regard to projects of which it would be desirable to obtain a motion picture record. Such a record of the outstanding researches is believed to be desirable and the bureau is considering plans for the purpose. By selecting a few of the more important investigations under way for picturization each year it is believed that a valuable photographic record of permanently interesting fundamental highway research will be gradually built up.

Questionnaires sent to the State contact men of the highway research board produced such a gratifying return that it has seemed desirable to compile and publish the data as a source of information in regard to current and recent projects. As it was emphasized in the request sent to the contact men that information was desired only with regard to projects suitable for picturization it is possible that some valuable information has been omitted from the reports.

The material is tabulated in accordance with the classification of the field of highway research arranged by W. K. Hatt, formerly director of the Highway Research Board.[1] Only those heads and subheads have been used which apply to the answers included in the questionnaire. Information was submitted by 39 State highway departments and 42 colleges and universities.

I. ECONOMICS:

1. Traffic studies—

United States Bureau of Public Roads in cooperation with States, etc.—Field work of traffic studies made in cooperation with the State and county highway departments has been completed in Connecticut, Maine, California, and Cook County, Ill. In cooperation with University of Tennessee and the State highway department a study has been completed in several counties of Tennessee. Studies are under way in Ohio and Pennsylvania in cooperation with the State highway departments. The data include traffic counts, net and gross loads of motor trucks, trip mileage, percentage of overloads, traffic in relation to population and other factors of value in determining highway budgets, selecting pavement types, and enacting traffic regulations.

California Highway Commission.—A traffic census is taken on all State highways at regular intervals, and this information is classified for future use. This work is under the immediate supervision of the maintenance engineer.

Colorado State Highway Department.—This State is now taking a traffic census on all the main highways. Counting stations are situated on the eastern and western slopes of the Rocky Mountain Divide.

Florida State Road Department.—This State has compiled a tabulation of traffic on the State highways.

Maine State Highway Commission.—A traffic census was taken during the period from July 1 to July 20, 1925, on the bridge between Kittery, Me., and Portsmouth, N. H. The object of this census was to determine the total amount of traffic coming in and going out of the State. The count showed the number of passenger cars, trucks, passenger busses, and horse-drawn vehicles. The registration by States was also taken.

Ohio Department of Highways and Public Works.—A comprehensive transport survey is now being conducted in cooperation with the Bureau of Public Roads. This work will cover a period of at least one year.

I. ECONOMICS—Continued.

1. Traffic studies—Continued.

South Carolina State Highway Department.—Traffic counts are being made at 117 stations for one day of each month. The object is to determine the most-used roads in order to select better types of surfacing.

University of Tennessee.—The department of highways and the University of Tennessee are engaged in making traffic studies throughout the State. Traffic counts are taken at typical stations four times each year.

Utah State Road Commission.—A traffic census is being taken this year. A count is being made on 13 days throughout the season beginning with May 6 and ending with December 1.

3. Cost of transport—

Vehicle—

Iowa State College.—Highway transportation costs. Engineering Experiment Station Bulletin No. 69.

4. Economics of location—

Iowa State College.—The Economics of Highway Grades. Engineering Experiment Station Bulletin No. 65.

North Carolina State College.—The determination of the power and cost of fuel required on State highways. Measurements are made on gasoline and electric trucks and cars with electric recording and integrating meters. The amount of the electric current and the speed are determined and from this the torque and power are calculated for different speeds on different roads and under different conditions. The test truck is calibrated for torque on the main driving shaft and on the driving wheels.

II. OPERATION:

1. Control of traffic—

Police regulations—

University of Pittsburgh.—The effect of automobile parking on traffic congestion. Observations are made of the speed of moving vehicles in the congested areas where different numbers of vehicles are parked.

7. Maintenance methods—

Routine—

Utah State Commission.—Field uses of bridge repair outfit. The unit consists of a 5-ton class B Liberty truck, on which is mounted a cement gun and an air compressor of 212 cubic feet capacity. The cement gun will be used for repairing concrete culverts and abutments which have been attacked by alkali. The truck also carries a compressed-air outfit for painting bridges and guard rails. Two jack hammers are part of the outfit. A car trailer hauls gas, oil, paint, tools, etc.

Replacement—

Colorado State Agricultural College.—A study is being made of joint fillers for use in maintaining concrete pavements.

Iowa State Highway Commission.—A study to determine the most satisfactory crack filler for use in concrete pavement maintenance. Observations were made on several sections of pavement on one road with the cracks in each section filled with a known commercial filler. The usual laboratory tests were made on each filler.

Michigan State College.—Laboratory tests are being made on 30 makes of bituminous fillers for cracks in concrete pavements. Observations are being made on 3 miles of road where these fillers are being used. The object is to secure information on which to base the revision of the specifications for crack filler.

Snow removal, etc.—

Michigan State Highway Department. Work has been completed on an investigation of snow removal problems. The data show that a definite relation exists between temperature and snowfall. The effect of this relation is also shown on the length of the snow removal season and the amount of snow which actually remains to be removed after taking into account the amount which falls at a temperature at which no snow removal is necessary. A definite rating for the various counties in the State has been worked out and a comparison is possible between suitable equipment and a reasonable cost of carrying on the work. The report will be published shortly by the State highway department.

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[1] The Field of Highway Research, by W K. Hatt. Public Roads, vol. 4, no. 5, Sept. 1921.

II. OPERATION—Continued.

7. Maintenance methods—Continued.

Snow removal, etc.—Continued.

Nevada Department of Highways.—The department has constructed a number of snow-removal units, one of them in particular being built on new lines. It is known that these plows and other lighter units are satisfactory in a depth of snow up to 18 inches. An attempt, however, has been made to build a snow plow that will handle easily 3 feet or more of hard snow. Since building units last year there has not been sufficient snow to make tests. If possible the tests will be made during the coming season.

Washington State Highway Department.—Various types of equipment are being tried out to determine the most economical method of removing the snow from the Sunset Highway in the Snoqualmie Pass through the Cascade Mountains.

Wyoming State Highway Department.—Investigations of the efficacy of elevating highway grades as a means of preventing the obstruction of roads by snow.

10. Safety—

Pennsylvania Department of Highways.—In 1924 guardrail tests were made. These are being continued this year. Loaded trucks are run into various types of guardrail to determine their resistance to impact and their capacity for saving life.

III. DESIGN (road):

1. Subsoil studies, properties—

Physical—

United States Bureau of Public Roads.—Intensive laboratory studies of the properties of subgrade materials are being made. These include studies leading to the proper classification of subgrade materials, investigations of their physical properties, and the development and standardization of tests for subgrade materials. Both laboratory and field tests are likewise being made in an effort to improve the supporting power of subgrades by manipulation or by the use of admixtures.

California Institute of Technology.—Tests are being made to determine the power exerted by confined adobe soils when the moisture content is increased from the dryness to saturation. The increases in volume from a dry condition to saturation are also being measured.

Colorado State Highway Department.—The department at this time is taking core-drill samples of pavements and samples of the underlying subgrade material. It is also sampling the subgrade soils on contemplated projects. These investigations are being made in cooperation with the Bureau of Public Roads.

Georgia State Highway Department.—Investigations are being carried on to determine the effective life of sand-clay, topsoil, and similar surfacing materials on the State highway system. Samples of surfacing material are taken annually from 29 Federal-aid projects which are under study. These samples are analyzed in accordance with standard methods and studies are being made on the several ingredients to determine the factors which control the stability of such soils under known traffic and weather conditions. Maintenance cost data are being kept on these projects and traffic counts are being made annually. These studies are expected to be carried on until the roads need to be reconstructed. At that time it is hoped that the combined economic and scientific data will be sufficient to determine the efficient use of these roads under known traffic density.

Iowa State Highway Commission.—An investigation to determine the cause and to devise a remedy for the subgrade failures occurring most frequently in cuts on earth and gravel roads in northwestern Iowa.

Michigan State Highway Department.—An investigation to determine the causes for the settlement of earth fills on unstable soils has been begun. An attempt is being made to determine the percentage of shrinkage probable on muck soil and the shape of the cross section of the earth fills in the muck. An attempt will also be made to determine the amount of muck beneath the fill which is compressed to such an extent that a comparatively stable support is provided for the fill above it.

Ohio Department of Highways and Public Works.—Soil investigations including laboratory tests are being made on experimental sections of pavements with specially prepared subsoils. The work is being carried on in cooperation with the Bureau of Public Roads and the Ohio Engineering Experiment Station.

III. DESIGN (road)—Continued.

1. Subsoil studies, properties—Continued.

Mechanical—

Iowa State Highway Commission.—The determination of the coefficient of friction of a concrete slab moving from rest and in motion on various soils. Concrete slabs, each 24 by 24 by 8 inches in size cast in place on the soil, are being studied. They are pulled with a hoist. The load is measured with a 1,200-pound spring balance and the movement observed and measured with a micrometer miscroscope.

Improvement by treatment—

Iowa State Highway Commission.—The effect of various finely powdered admixtures incorporated with the surface soil of earth roads. A total of 3½ miles of road was treated, one-half of the mileage on a tough, clay soil and the remainder on Missouri Valley loess. Portland cement and hydrated lime in various amounts were used on different sections of each road. They were mixed with the earth to a depth of 6 inches.

University of Illinois.—A laboratory investigation attempting to determine the basic principles involved in the construction of oiled earth roads. This includes a study of both the oils and the soils. Although considerable work has been done it has not yet reached the stage of publication. The work will continue during the coming year. A graduate research fellowship for the study of earth roads has been established. The work will cover a period of about two years.

University of Missouri Engineering Experiment Station.—A study to determine the effect of lime on earth roads. Commercial hydrated lime is incorporated in the top 6 inches of the soil of the road to determine the beneficial effect. It is hoped by this treatment that the maintenance of such roads will be made easier, particularly with respect to dragging and prevention of the formation of ruts. Some sections have been tried with a 2-inch depth of application of the lime and also with varying percentages of lime. Laboratory tests have been carried on in conjunction with field tests to determine how the soil is affected by the addition of the lime.

Effect of climate—

Minnesota State Highway Department.—A study of the occurrence of frost upheavals in the spring, the locations at which they are most likely to occur, and an investigation of methods of prevention.

2. Base course—

Ohio State University Engineering Experiment Station.—The method of maintaining the smoothness of a granular subbase beneath a concrete pavement while paving materials are being brought to the work, mixed, and placed.

3. Surface—

Character—

Alabama State Highway Department.—Determination of a clay-asphalt mixture for surfacing. It is desired to produce a serviceable and economical surfacing by the admixture of asphaltic oils with clay, sand, gravel, etc., adjacent to the roadway.

Connecticut State Highway Commission.—A study to determine an economical method of laying a bituminous surface on a concrete road with a pitted surface. Experiments are being carried out on sections of pavement surfaced with various kinds and depths of bituminous materials.

Michigan State Highway Department.—An investigation of the various types of rigid pavement now in service on the trunk-line and Federal-aid highway system in an effort to determine what methods of reinforcement of the slab, subgrade treatment, etc., have been particularly effective since their construction during the past five or six years. A number of projects are available on which experimental work has been done in the past few years.

Wear by elements—

Illinois Department of Public Works and Buildings.—An investigation to determine the temperature through pavement slabs and a study of the resulting stress and the curling action of the slab.

Wear by traffic—

United States Bureau of Public Roads.—Laboratory and field studies to determine the characteristics and properties of bituminous paving mixtures, especially with regard to the determination of the cause of longitudinal displacement or shoving. Field experiments

III. DESIGN (road)—Continued.

3. Surface—Continued.

Wear by traffic—Continued.

are being carried out on a circular track with concrete base surfaced with short sections of widely different mixtures. Traffic has been imposed by means of a loaded truck and the relative stability of the various sections obtained by recording the displacement of plugs set in the surface.

Laboratory tests have been made to determine the properties of materials used in the field test, and a new modified shear test has been developed which appears to measure the resistance to displacement of sheet asphalt and bituminous concrete mixtures.

Tractive resistance—

Iowa State College.—Resistance to the translation of Motor vehicles. Engineering Experiment Station Bulletin No. 64. Tractive resistance and related characteristics of roadway surfaces. Engineering Experiment Station Bulletin No. 67.

5. Impact—

United States Bureau of Public Roads, Society of Automotive Engineers, and Rubber Association of America.—The objects of this cooperative investigation are the determination of the effect of motor trucks operating on highways as influenced by various truck, tire, and road factors, such as type of tire, design of truck, and character of road surface; and the measurement of stresses in road surfaces as influenced by motor-truck wheel impact. Experiments so far include the determination of the cushioning of pneumatic, cushion, heavy-duty cushion, high-profile, non-skid, and regular types of solid tires. Tests have been made with both new and worn-out tires, and with new tires cut down to various heights to simulate the effect of partly worn tires. Both static load and impact tests have been made with trucks equipped with various tires, as well as tests on actual roads, one test road section being selected to represent a smooth and the other a rough surface.

8. Reïnforcing—

Highway Research Board, National Research Council.—A nation-wide survey is being made to determine the value of reinforcing in concrete pavement. Pavements in good and bad condition are being compared to determine the relative value of the different types of reinforcement. In addition to differences in the subgrade conditions notes are being taken on the age, mix, traffic and cross-section variations.

IV. DESIGN OF VEHICLES (as related to the road):

1. Design of vehicle—

University of Kansas.—A study of the air resistance to motor vehicles. A wind tunnel is used in which the car to be tested is placed on a feebly swinging platform. As the wind is forced past the car its effect is observed and the force recorded

3. Surface—

Wear on tires—

University of Kansas.—A study to determine the relative effect of different types of pavement on the wear of motor-vehicle tires. A Dodge touring car and a laboratory apparatus using regulation-size tires are used in the tests.

4. Alignment—

Grades—

Michigan State Highway Department.—An investigation of the behavior of 1½-ton trucks on different types of surfaces and on various grades has been completed. This takes into account all factors at the various speeds. The factor which is new and which is believed never to have been considered before is that of the moment of inertia of the rotating part of the motor vehicle including the wheels, transmission system and engine.

6. Safety—

Iowa State Highway Commission.—The determination of various methods of securing satisfactory driving lights for automobiles and the regulation of the use of these lights.

V. CONSTRUCTION:

1. Materials—

Bituminous—

Iowa State Highway Commission.—The effect of various preservative treatments for wood posts used in guard-rail construction. Several sets of posts were treated by different methods and installed along the roads in three separate counties. Observations were made of ordinary installations, and laboratory tests were carried on for the various treating fluids.

North Carolina State Highway Commission.—The determination of the proper thickness of a sand-asphalt wearing surface under various traffic conditions. The observations were made on pavement surfaces of 1½ and 2 inch depths under known traffic conditions.

South Carolina State Highway Department.—The observation of bituminous treatment of earth roads. Forty miles of experimental roads with various tar and oil treatments have been constructed. The object is to develop a satisfactory and economical surfacing for earth roads, such as topsoil and sand-clay.

Nonbituminous—

United States Bureau of Public Roads.—An investigation to determine how the character and type of aggregate affects the resistance of concrete road surfaces to wear produced by traffic loads. Sixty-two sections of cement-concrete surfacing were constructed in the form of a circular track 600 feet in circumference. Various types of coarse aggregate, such as crushed stone, gravel, and slag were employed, each of which varied in quality from the best obtainable to materials which have been considered of questionable value. These test sections have been subjected to the action of a motor-driven testing machine equipped with solid rubber tires designed in such a manner as to produce the same traffic effects as an actual motor truck. An amount of traffic equivalent to 300,000 applications of a single wheel load operating at a speed of 22 miles per hour and loaded to 600 pounds per inch width of tire, was first put upon the pavement. This was followed by a second test in which the testing machine followed an entirely different path and in which traffic was placed upon the pavement equivalent to 75,000 applications of the same wheel load, with the exception that the tires were equipped with nonskid chains.

A series of concrete tests is now being conducted in which the coarse aggregate varies both as to character and quality. About the same range of quality and character of aggregates is being employed in this test as was used in connection with the tests for wear-resisting properties of concrete. The specimens are subjected to alternate freezing and thawing in such a manner as to simulate the effect of actual frost action in the field. The effect of frost action is determined both by visual examination of the specimens and by subjecting them to flexure tests.

California Highway Commission.—A study to determine the efficiency and cost of calcium chloride for the prevention of dust on a heavily traveled gravel road has been made in one of the divisions of the State.

Colorado State Agricultural College.—A survey is being made to determine the location in the State of road materials for various types of highway construction. Samples of the materials are tested in the laboratory.

Another study is being made to determine the effect of beet pulp on concrete and mortar.

Iowa State Highway Commission.—A study to determine the effect of shale in various amounts in the aggregate upon the strength of concrete. This includes a study of rotten stone. Compression tests are made on 6 by 12 inch cylinders and transverse tests on 4 by 8 by 30 inch beams.

A study to devise methods of using poorly graded sands to produce concrete equivalent in workability and strength to normal materials. The mortar-void theory of proportioning was tried. The specimens consisted of briquettes and 2 by 4 inch cylinders.

A study to determine what physical characteristics of Iowa limestones affect the quality of concrete made by using these limestones as the coarse aggregate and to devise tests which would consistently indicate variations of those characteristics. Abrasion tests were made in the Deval machine with standard and slotted cylinders.

A series of tests of large-sized concrete pavement slabs constructed by the vibrolithic process and corresponding slabs constructed in the conventional manner. Both kinds of slabs were tested for modulus of rupture by means of a specially constructed testing machine. Various proportions of cement and aggregate were used in the slabs, one-half of which were tested with the top surface in tension, the balance with the bottom in tension. Tests are being made at the age of 28 and 90 days and one year.

Later this test was continued with a new operator to determine the durability of the limestone. Accelerated weathering tests and soundness tests were made. Particular studies were made of the steam-sulphate test and of the freezing and thawing tests.

Later this work was continued by making a particular study of the soundness, absorption, specific gravity, and standard Deval abrasion tests.

A study to determine the effect of varying ratios of fine and coarse aggregate in concrete mixes upon the strength and resistance to wear of the concrete. This is a corroboration of Crum's method of proportioning. Proportions were made by weight. The specimens made up in 6 by 12 inch cylinders and 4 by 8 by 30 inch beams were subjected to the usual tests. Both types of specimens were made in the field and in the laboratory.

A study to determine the serviceability of various paint mixtures under service conditions.

A study to determine the effect of the various chemicals found in the natural waters of the State upon the strength of the concrete mixed with these waters.

A study to determine the physical or chemical properties of a soil or clay which make it unsuitable as binding material for use in gravel road construction. Methods of tests were used as proposed in a paper on "Physical properties of subgrade materials," read by J. R. Boyd of the Bureau of Public Roads before the twenty-fifth annual meeting of the American Society for Testing Materials.

A study to determine the thermal coefficient of expansion of concrete made from different mixes.

An investigation of the characteristics of a serviceable paint film. Microscopic photographs of the surface and cross sections of the various paint films were made.

A study to determine the effect of the size and shape of concrete test beams upon the modulus of rupture. Beam specimens 4 by 8 by 30 inches and 4 by 4 by 60 inches in size were tested at different span lengths and in different positions.

A study to determine a method of using Platte River sand-gravel to produce concrete having strength equivalent to that produced by concrete made from a typical Iowa material possessing similar characteristics. The usual laboratory specimens were made. These consist of beams and cylinders and a special effort was made to develop a technique for measuring the workability of plastic concrete.

A study to determine the properties of the high alumina cement recently introduced into the United States. Tests were made on laboratory specimens consisting of standard briquettes and small cylinders.

John Hopkins University.—A study to determine the relation, if any, between static and impact strains in concrete. Concrete beams 6 by 8 by 4 feet long are subjected to impact loading by means of a rubber-cushioned weight which is allowed to fall upon the beams at their third points. The unit deformations are measured with graphic strain gauges inserted in both sides of the beams, six on each side arranged vertically one over the other at the mid point of the beams. The variable impact forces are determined by an accelerometer. Deflections of the beams are also measured. Corresponding measurements are also made for static loading which is applied by a standard testing machine.

Kansas State Agricultural College.—A study to determine the abrasive resistance of concrete with various kinds and proportions of aggregate. Nine-inch concrete spheres are cast and for 90 days are tested in a standard brick rattler. At the end of the test period, four ½-inch cores are drilled from each sphere and tested in compression.

Michigan State College.—A study to determine the effect of low temperatures on the time of setting and the strength of Lumnite cement. The compression specimens were exposed to cold weather of different degrees at various ages and with different coverings.

A study to determine the bond between Portland cement and Lumnite cement. The bond was tested in shear and the effect of wear was observed on a top coat of the Lumnite cement on a pavement with a Portland-cement base.

Minnesota Highway Department.—A study to determine the effect of shale on the compressive strength of concrete. Concrete cylinders, 6 by 12 inches in size, containing varying percentages of shale were subjected to alternate freezing and thawing. The object was to determine the quantity of shale which should be permitted in coarse aggregate for Portland cement concrete.

A study of the limestone quarries in the State of Minnesota which are operated on a commercial scale. This report consists of geological descriptions of the various quarries in the State and physical and commercial tests and samples of rock taken from each quarry.

A study to determine the soundness of Minnesota stone. The tests were made in accordance with the methods described on page 8 of United States Department of Agriculture Bulletin No. 1216. Tests were made on 4 samples of trap rock, 5 samples of quartzite, 2 samples of granite, 14 samples of limestone, and 1 sample of sandstone. All samples were subjected to 20 immersions in copper sulphate. The fineness modulus was determined on each sample before and after tests were made. In this way the percentage of loss was determined.

A study to determine the effect of weather on the compressive strength of concrete. This involved alternate freezing and thawing, especially of concrete containing shaley or laminated limestone.

A study to determine the effect of moisture content on the volume of sand and the effect of the grading and character of the coarse aggregate on the percentage of voids.

A study to furnish comparative data on vibrolithic pavements and one-course concrete pavements constructed under similar conditions.

A study to determine the merits of the patented pavement known as "raw hide." The data were obtained from a short experimental stretch of this pavement constructed during the fall of 1924.

A study of the effect of time on the compressive strength of Lumnite cement concrete and observations on Lumnite cement concrete placed during freezing weather.

A study of the merits of Willite pavement. This investigation includes tests on the various grades of asphalt such as Texas, Mexican, Bermuda, Trinidad, etc., with and without copper sulphate. Tension, toughness, sag, and impact tests were made on sheet asphalt mixtures containing copper sulphate.

A study of an experimental road consisting of sections of Portland cement concrete with various kinds of aggregate and types of reinforcement. One section consisted of Lumnite cement concrete and another of sheet asphalt with an asphaltic concrete base.

A study to determine the value of marl as a binder on sandy roads. The experimental roads were built in 1922. A report of this test was published as Bulletin No. 1 of the Engineering Experiment Station of the University of Minnesota.

New Jersey State Highway Commission.—Deterioration surveys have been made of the concrete pavement on the State highways. The results will be compiled and compared with designs, construction conditions, and maintenance costs.

Purdue University.—The fatigue of concrete. Concrete specimens are subjected to repeated stresses or reversals of stress in a specially constructed apparatus. The object is to determine the fatigue limit of concrete under repeated loads.

Tennessee Department of Highways.—A study to determine the tensile and compressive strength of Lumnite-cement concrete at ages varying from 11 hours up to 12 months. The tensile and compressive specimens were 2 by 4 inches in size.

A field study to determine the behavior of rock asphalt as a road-surfacing material.

V. CONSTRUCTION—Continued.

1. Materials—Continued.

Nonbituminous—Continued.

Texas Agricultural and Mechanical College.—A study to determine the suitability of mudshell as an aggregate for bituminous pavements. Numerous specimens were prepared from various formulas and tested for stability by means of an improvised mold and plunger and by true punching shear. Comparisons were made with four other types of hot mixture.

University of California.—A study to determine the expansion of concrete due to the absorption of moisture. These tests have been under way since the first of the year. A room has been constructed in which the temperature and humidity may be maintained at any desired temperature and 3 by 3 by 40 inch specimens have been observed periodically to determine the rate of change in weight and length due to variations in the moisture content.

A study to determine the fatigue of concrete. Beam specimens, 4 by 6 inches by 6 feet, are subjected to repeated center loads up to 100,000 repetitions.

Tests are being inaugurated to determine the plastic deformations of concrete under continuing load. These tests will be made to determine a rate of flow of concrete in compression and also in flexure.

Fundamental mechanical properties—

California Highway Commission.—In order to determine the relative smoothness of various pavements, vialog records have been made of most of the existing pavements on the State highway system. New records are being made from time to time to determine what changes, if any, occur in the condition of the pavement. Records are also being made of new pavements to ascertain the relative efficiency of the finishing methods and construction operations.

Illinois Department of Public Works and Buildings.—A study to determine the effect of impact on pavement slabs by equivalent static loads through the medium of slab deflections.

Tests have been made with the Illinois fatigue machine to determine the fatigue stress in concrete.

Methods of tests—

U. S. Bureau of Public Roads.—Extensive laboratory and semifield tests are being conducted for the purpose of determining the most satisfactory method of measuring the consistency of Portland cement concrete. Comparative tests have been made, using the slump test, the flow test, and a special device known as the *plate test* developed by the bureau. All of these tests have been conducted with a view of developing a method which will control the water-cement ratio and, therefore, the strength of the concrete.

Illinois Department of Public Works and Buildings.—A number of pavement surfaces have been profiled with a profilometer in order to study the roughness of the slab.

Iowa State Highway Commission—A study to determine the effect of the number of pieces of rock used in the abrasion test for the French coefficient of wear.

A study to determine the effect of a standard shot rerun of the abrasion test.

A brief study to determine the effect of the number of revolutions in the determination of the French coefficient of wear.

An investigation of the various proposed methods for making quantitative analysis of the quantity of shale in gravel. Flotation in heavy liquids and separation by handpicking was tried.

An investigation of the various adaptations of the Deval test and others with the Deval machine.

Investigations of the abrasion tests of sand to determine the suitability of these tests for fine aggregates.

An investigation to devise a test for cement which will give consistent results in a shorter period of time than that required by the standard tests now in use. Standard briquets and 2 by 4 inch standard specimens were used.

An investigation to determine the adequacy of the standard Deval abrasion test for establishing the suitability of the mortar portion of the various concrete mixes used in highway work.

An investigation to devise laboratory tests which will consistently and accurately determine the characteristics which govern the behavior of a lubricating oil under known conditions of service. The work has consisted of a brief study of the methods now used for testing lubricating oils and a series of these tests was made upon a large number of commercial oils.

V. CONSTRUCTION—Continued.

1. Materials—Continued.

Methods of tests—Continued.

An investigation to devise a method of testing rotten stone, which, when used by different operators, will consistently differentiate this undesirable material.

Massachusetts Institute of Technology.—A study to determine the effect of slotted cylinders of the Deval machine.

Preparation and treatment—

Iowa State Highway Commission.—A study to determine the causes of and methods for controlling the hair cracks in concrete-pavement slabs. Small slabs were built upon various types of subgrade soil. Each soil was subjected to a similar set of conditions.

An investigation to determine the effect of varying sand ratio, varying consistency, and varying curing period upon the modulus of elasticity of concrete.

Wisconsin Highway Commission.—A study to determine the best method for the surface treatment of gravel roads from which has been developed the Wisconsin special-mix method which has been used in this State during the past three seasons.

Proportioning—

Illinois Division of Public Works and Buildings.—A study to determine the bulking effect of materials of different gradations by means of the Illinois bulkmeter.

Iowa State Highway Commission.—A study to determine the specific gravity of sand and gravel from a large number of deposits for use in the weight method of proportioning.

A study of the various methods for determining the cement content and the proportions of aggregate in cured concrete. Mechanical separation, chemical analysis and microscopic observation and measurement were used on prepared specimens wherein all the constituents were accurately known.

An investigation to determine the effect of the curing of the coarse aggregate upon the strength of concrete made in different proportions. Specimens consisted of 6 by 12 inch cylinders for the compression tests and 4 by 8 by 30 inch and 8 by 8 by 30 inch beams for the transverse tests. Workability of the plastic mix and shrinkage of the curing concrete were given special observation.

North Carolina State Highway Commission.—A study to determine the accuracy of proportioning fine aggregate by inundation. Several miles of concrete pavement were constructed with fine aggregates proportioned by loose volumetric measurements. Adjacent to this several miles of concrete pavement were built in which the same materials were proportioned by the so-called inundation method. Points observed are the effect on the cement, constant plasticity and workability of the concrete, and strength of the concrete as determined by cores taken from the completed pavement and broken in a standard testing machine.

University of Tennessee.—It is hoped that opportunity will be afforded to make during the next two years a study of fine dust as asphalt-pavement filler. It is hoped that the project will be under way by January of next year.

2. Mixing—

Efficiency of mixer—

Iowa State Highway Commission.—An investigation to determine the effect of the vibrolithic process as applied to the finishing of concrete-pavement slabs. Ten slabs, each 7 feet wide, 8 inches deep, and 75 feet long were constructed using five different proportions and 2 slabs for each proportion, 1 of plain and 1 of vibrolithic concrete. The slabs were divided into 750 beams 12 inches wide, 8 inches deep and 7 feet long. Transverse tests were made on the beams and compression tests on the cores drilled from the beams.

3. Placing—

California Highway Commission.—A field study is now being made to determine the relative efficiency of various periods of water curing. Sections of pavement were cured with earth covering kept wet for varying periods. Cores are being drilled at various ages. The last of these cores will be drilled in June 1926, at an age of 1 year.

V. CONSTRUCTION—Continued.

3. Placing—Continued.

North Carolina State Highway Commission.—A study of the methods of curing concrete pavements by moist earth and ponding, by calcium chloride sprinkled on the surface of the pavement and by calcium chloride (or Cal) used as an admixture. The points to be noted are the results of the core breaks, the visual inspection of the finished pavement surface and the character of concrete during construction.

A study to determine the necessity for joints in concrete pavements. A 10-mile plain concrete project will be built, 5 miles of which will be constructed without expansion joints and 5 miles with expansion joints. As an alternative to this method of construction, alternate sections 1-mile long may be built with and without joints. It is not known when this investigation will be begun.

4. Methods of testing roads—

Cores—

Minnesota Highway Department.—An investigation to determine the effect of capping concrete cores on the compressive strength of concrete, and also to determine what correction factors, if any, should be used for cores of various types.

6. Drainage (and drainage structures)—

U. S. Bureau of Public Roads.—In order to measure experimentally the distribution and intensity of abutment reaction in skew arches, a series of tests has been conducted on concrete arches, one-fourth actual size, constructed on 30°, 45°, and 60° skews. The arches were tested under a uniform load applied through 42 symmetrically placed loading points on the arch ring. Abutment reactions were measured by means of the soil pressure cells developed by the bureau, which were so placed as to take both the vertical and horizontal reactions at one end of the arch.

California Highway Commission.—A more or less elaborate study of corrugated-iron culverts is to be made during the latter part of this year with a view of determining, if possible, the relative durability of pure iron and of covered steel pipe.

Iowa State Highway Commission.—An investigation to determine the laws governing the behavior of steel rollers of varying temperature under loading when in compression. The preliminary work was done on rollers up to 48 inches in diameter. The project is now being carried on by the Engineering Experiment Station of the Iowa State College at Ames, Iowa.

A study of the factors affecting the design of concrete culvert pipe has been made in cooperation with the Engineering Experiment Station of the Iowa State College at Ames, Iowa. The station is now carrying on this work.

Maine State Highway Commission.—An investigation is being made of all classes of culvert installation. The study for the most part is being confined to culverts which have been installed at least seven or eight years or longer. The work will cover all sections of the State.

South Carolina State Highway Department.—The effect of sea water on reinforced concrete and vitrified-clay culvert pipe. The specimens are placed between high and low tide in the sea water at Charleston, S. C., and observations are made of the effect on the materials.

Tennessee Department of Highways.—A field study of the condition of several types of pipe culverts and also small concrete box culverts and stone culverts.

University of North Carolina.—A study to determine the vertical earth pressures on culvert pipes with varying heights of fill up to 20 feet. The completed tests will include both sand and clay as a filling material. Beginning in the summer of 1925 the tests will be continued using pipe of various sizes and kinds. In 1926 it is hoped to determine the radial pressure as well as the vertical pressure of the soil on the culvert pipe.

University of Pittsburgh.—Tests have been made on various sizes of concrete-culvert pipe to determine their strength. The pipes were tested to destruction and the work was done in cooperation with the Concrete Products Co.

8. Reinforcing, handling, and placing—

Illinois Division of Highways.—An investigation is being made as to the properties of rail steel and its reaction under impact when such steel is used for concrete-pavement reinforcement.

Iowa State Highway Commission.—A study of the behavior of reinforcing steel in concrete pavements. Special attention was directed to bond stress and the behavior of steel at cracks in the pavement. Preliminary work has been done on the testing of steel of different grades of hardness.

(Continued from page 206)

for 16 to 24 hours and then with earth kept wet for 7 days may safely be opened to traffic in 14 days.

3. Curing with a surface application of flake calcium chloride, 2½ pounds per square yard, gave strength ratios of from 88 per cent at 7 days to 83 per cent at 90 days; with less than this amount and with 3 and 5 pounds there was a slight reduction in strength.

When the calcium chloride was washed off after 3 hours, the strengths were reduced perceptibly; washing off calcium chloride after one day gave essentially the same strengths as when left on the surface.

Surface hardness for calcium chloride cured beams was considerably less than for beams cured with wet earth or with asphaltic paper.

4. For beams molded and cured in concrete forms using calcium chloride, 2½ pounds per square yard, the strength ratios ranged from 100 per cent at 14 days to 89 per cent at 90 days. The strengths in this case were about 12 per cent higher than for beams molded in wood forms and cured with a similar amount of calcium chloride.

5. Asphaltic paper curing gave average strength ratios ranging from 92 per cent at 7 days to 78 per cent at 90 days; surface hardness was almost as high as for wet-earth curing.

6. Both air curing and sodium-silicate curing showed low strength and surface hardness. The strength ratios for these methods of curing ranged from about 77 per cent at 7 days to 74 per cent at 90 days.

7. Indentation loads used in measuring surface hardness averaged 21 times the modulus of rupture; the greatest surface hardness was found for concrete of highest flexural strength. This method of test showed that calcium chloride, sodium silicate, and air curing produced a more friable surface than wet earth or paper curing.

EARTH AND GRAVEL ROAD BIBLIOGRAPHY PREPARED

On account of the great interest in secondary highways the Engineering Societies Library has prepared a bibliography of books and articles on earth and gravel roads, published between January, 1920, and June, 1925.

The list, designated as S. 4085, contains 140 references with brief annotations. Mimeographed copies may be obtained for $1.50 each by writing to the Engineering Societies Library, 29 West Thirty-ninth Street, New York City.

HIGHWAY LANTERN SLIDES AVAILABLE

The Bureau of Public Roads has recently placed in its district offices at Portland, Oreg., San Francisco, Calif., Ogden, Utah, and Denver, Colo., sets of lantern slides illustrating the methods of constructing the principal types of roads. The slides are in six sets and are intended for use in engineering schools and colleges. Complete information with regard to the slides and how to obtain them can be secured from the Washington office or the above district offices of the bureau.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
*347. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463. Earth, Sand-Clay, and Gravel Roads. 15c.
*532. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583. Report on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*586. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1916. 10c.
*660. Highway Cost Keeping. 10c.
670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691. Typical Specifications for Bituminous Road Materials. 10c.
*704. Typical Specifications for Nonbituminous Road Materials.
*724. Drainage Methods and Foundations for County Roads. 20c.
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
No. 1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1259. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULAR

No. 94. TNT as a Blasting Explosive.

FARMERS' BULLETIN

No. *338. Macadam Roads. 5c.
*505. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *727. Design of Public Roads. 5c.
*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

252.A

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

OCTOBER 31, 1925

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5.970.097.71	$ 2.863,197.86	611.8	$ 5,290.661.84	$ 2.496.564.59	293.9	$ 10.322.000.26	$ 5.031.107.84	535.4	$ 378.592.64	$ 175.050.00	11.6	$ 2.243,745.71	Alabama
Arizona	9.580.133.43	5.016,119.94	613.8	237.926.80	145,397.06	7.1	2,051.812.12	1.308.811.30	166.1	176.977.10	108.150.70	26.7	1.982,862.00	Arizona
Arkansas	13.310,190.08	5.380.181.73	1048.9	2,641.387.65	1.118.078.04	93.2	5.904,602.02	2.825.489.13	381.9	1,164,378.38	549,903.60	96.4	464,244.50	Arkansas
California	22.346.175.99	10.719.249.61	894.8	1.758,210.30	782.842.66	40.4	10.448.983.86	5.261.917.12	319.5	1.203.924.54	489,986.35	39.3	2,344,113.26	California
Colorado	11.876.703.94	6.067,814.34	651.2	637.732.66	338.922.67	25.0	4.527.389.17	2,257,693.88	221.9	338.945.90	190.092.34	35.3	2.090,904.77	Colorado
Connecticut	4.558.639.29	1.819,368.66	101.6	750,865.91	250.212.14	13.4	1.713.128.78	549.559.48	26.9	204,505.29	48,608.15	2.3	1,192.504.57	Connecticut
Delaware	4,281.559.81	1.496,190.55	107.1	455,744.13	212.544.95	12.3	980.042.66	388.784.75	22.6				10,912.65	Delaware
Florida	2.969.273.72	1,405.487.97	96.3				8.970.318.13	4.370.709.60	261.7	1,991.337.04	650,530.77	41.5	761,040.66	Florida
Georgia	20.156.002,37	9.406,366.46	1478.3	2,039,093.60	991,054.44	121.0	11.092,111.41	5.226.115.36	705.1	1.170.995.66	483.749.27	87.9	343,478.47	Georgia
Idaho	9.394.676.80	4.815,332.26	600.1	176.117.58	101.367.48	21.0	2.750.264.23	1,733,255.24	177.0	933,074.03	562.087.44	41.2	410,995.58	Idaho
Illinois	40.010.481.10	18.640.076.28	1236.2	688.967.89	340.741.22	25.0	8.644.745.27	4,189,261.40	290.9				3.486.503.10	Illinois
Indiana	13.639.172.65	6.562.465.68	422.1	1.132.976.57	544.542.84	41.2	17.584.862.01	8.240.153.23	479.8	832,279.23	413.698.33	25.8	507.614.92	Indiana
Iowa	27,272,286.21	11.107.492.99	1996.9	722.462.52	320,365.06	36.5	7.272.120.64	3.266.326.98	457.5	1.480.053.73	699.848.90	144.3	2.032,060.07	Iowa
Kansas	26.399.695.77	9.756.273.32	831.4	2.784.450.08	1,213,748.87	146.4	10.670.447.84	4.512,216.72	517.9	1,787.131.43	505.704.68	104.6	1.405.301.41	Kansas
Kentucky	14.832,324.28	6.205.994.59	584.9	1.656.093.80	663.291.14	46.9	7.871.365.04	3.611.429.76	310.0	517.209.81	258.604.89	27.3	1.156.679.62	Kentucky
Louisiana	11,939.424.97	5.279.870.86	927.6	875.723.58	411.312.04	65.0	2.772,603.62	1,325,240.49	108.8	582.979.67	290.987.40	21.9	964,233.21	Louisiana
Maine	8.174.281.31	3.907.870.33	281.4	173.275.15	85.733.59	6.6	1.311.335.31	551.480.38	42.2	976,766.16	394,636.81	35.6	841,532.89	Maine
Maryland	8.132.506.90	3,849,383.15	294.4	368.826.12	177,191.15	14.7	2,814.750.06	1,244,591.15	126.3				19.267.55	Maryland
Massachusetts	14.047.656.22	5.467.661.28	300.6	505.080.98	165.015.60	11.0	6.155,325.67	1.629,813.05	86.9	523,237.49	173.035.13	8.1	1.584,145.94	Massachusetts
Michigan	16.234.000.80	7.328.316.91	612.6	5.227,433.17	2.405.657.00	208.7	12.044,705.07	5,523,127.94	328.5	433.069.40	158.370.99	0.3	2,709,474.16	Michigan
Minnesota	30.415.685.89	12.738.642.04	2721.2	3.550.542.83	1.506.322.07	279.5	7.353.175.06	2.888,700.00	608.9	1.023,293.92	171.600.00	44.9	156.347.89	Minnesota
Mississippi	10.292.285.79	4.988,702.73	803.4	1.134.270.59	566.407.24	83.6	8,337,386.37	4,162,437.53	437.0	1.083,118.42	541.559.18	61.1	575.708.32	Mississippi
Missouri	17.368.156.57	8.219.411.43	1118.9	2.830.765.80	1.363.795.71	121.5	25.921,368.10	10.264.311.41	763.8	1.362.237.84	115.523.89	32.3	416.546.56	Missouri
Montana	10.156.600.41	5.317.523.15	921.6	607,162.46	474.072.98	57.9	1.849.094.85	1.344.221.16	186.3	1.003.571.06	544,458.41	117.9	4.192,949.30	Montana
Nebraska	9.306.374.36	4.389.523.50	1570.6	1,362,105.81	660.957.40	117.0	8.761.990.25	4,347,216.75	887.7	2,420,926.59	1.190.961.25	213.4	2,458,438.10	Nebraska
Nevada	4.917,465.69	3.088.299.78	357.3	176.699.88	131.175.94	18.2	5,394,643.05	4.528,235.45	468.2	44,370.41	38,921.72	5.6	60,264.11	Nevada
New Hampshire	4.165.687.86	1,986.226.87	208.1	126.519.20	56.312.01	5.0	1.265.074.57	593,106.22	40.4				168.221.90	New Hampshire
New Jersey	11.961.357.45	3,820,679.99	219.1	870,702.97	207.502.78	10.3	9.870.567.74	3.453.916.40	81.2				50,612.83	New Jersey
New Mexico	8.717.999.18	4,914,070.61	1081.3	2.455.289.19	1.651.290.55	212.1	2.346,169.27	1.460.967.04	216.2	272.777.67	190.930.39	17.4	1.567,863.41	New Mexico
New York	28.597.769.67	12.229.076.63	831.5	3,991,290.64	1.562.381.59	100.1	33.598.825.75	10.621.551.55	661.6	7.123.300.00	1.746.285.00	116.5	4.238.734.33	New York
North Carolina	21.014.450.41	8,746.454.59	1119.8	2.371,883.81	1.007,858.84	67.1	8.215.648.74	3.469.733.08	199.4	1.787.907.60	762.380.84	45.0	22.224.65	North Carolina
North Dakota	10.829.263.82	5.268.930.47	1917.5	590,070.30	294.904.62	142.2	4,135.034.60	2,087,553.65	526.1	920,498.80	442.127.40	91.4	1.461,422.86	North Dakota
Ohio	41.572.252.81	15.244.993.93	1191.1	1.854.218.12	649,259.37	56.7	12.171,583.58	4.626,722.20	373.6	1,938,733.00	626.940.22	47.0	1,807,843.28	Ohio
Oklahoma	20.787,024.94	9.672.890.34	852.2	1.558.181.95	698.604.70	57.4	7.116.638.78	3,399,770.16	333.5	811.141.63	275.053.59	31.5	261,223.21	Oklahoma
Oregon	14.388,138.70	7.142.364.63	794.6	751.059.58	389.368.05	34.5	3.232.666.00	1.996.373.99	170.9	508.820.95	137.767.25	23.1	130.528.18	Oregon
Pennsylvania	43,054.935.19	16.222,023.97	850.3	2.667.611.93	834.606.72	49.5	36.004,538.70	9.640.406.67	651.5	2.463.989.12	826.309.13	61.8	468.514.51	Pennsylvania
Rhode Island	2.628,496.20	1.119.688.09	64.8				2,543.268.84	681,939.59	37.3	263.012.76	63.135.00	4.2	437.191.32	Rhode Island
South Carolina	11.163.347.84	5.121.267.54	1235.9	1.061.972.51	357.678.74	58.5	6,010,468.96	2.824.091.69	328.9	957.481.91	378.231.95	61.5	68.261.08	South Carolina
South Dakota	12,091.434.67	5,989.879.00	1447.9	1.535.493.16	743.311.55	192.2	6.588,931.26	3.107.824.85	936.1	86.427.58	48,425.04	14.0	55.151.56	South Dakota
Tennessee	13.799.140.98	6.732.079.77	497.9	2.565.128.53	1,261.638.03	84.4	9.992.630.24	4.427.733.01	337.9	241.502.80	119.851.60	11.3	1.120.969.59	Tennessee
Texas	54.120.970.93	21.057,940.12	3907.1	5.787.896.50	2.328.239.80	327.9	21.983,492.49	9.586,100.94	1265.5	4.914,240.46	2.212.696.79	323.1	994.536.35	Texas
Utah	6.259.159.41	3.818.836.91	423.1	132.178.99	99.980.35	5.9	3.224.624.68	2.193.626.23	255.5	325.575.07	246.654.73	16.9	611.529.78	Utah
Vermont	3.016.174.51	1,452,894.45	107.9	178.568.08	89.218.34	3.3	1,906.002.42	841.367.56	43.0	77.750.79	38,875.39	3.3	480.526.26	Vermont
Virginia	13,099.720.01	6.271.998.20	675.2	4.061.685.06	1.923,691.94	162.2	7.223,621.99	3.162.998.69	229.9	1.415.814.00	607.931.27	51.7	89,142.00	Virginia
Washington	13,352,504.18	6.117.211.97	526.7	1.964.825.31	855.987.39	87.7	2.426.786.31	1.115.400.00	74.9				927,196.74	Washington
West Virginia	7.343.200.96	3.230.293.33	325.7	252.664.30	123.013.99	12.9	6.774.986.54	2,719.659.25	179.1	981,982.41	367,090.29	35.2	118,519.24	West Virginia
Wisconsin	21.907.140.91	8,919,640.62	1461.7	107,365.47	53,526.17	6.6	5,850.782.11	2.876.686.58	281.4	621.435.40	312.333.00	40.5	3,406,366.63	Wisconsin
Wyoming	8,909,819.33	4,739,096.57	982.0	854,988.57	521,673.58	80.3	3.408.927.94	2.141.606.24	221.4	73,606.61	47.255.00	9.9	181,048.51	Wyoming
Hawaii							342,277.22	97,440.00	6.5				637,088.00	Hawaii
TOTALS	740,140,790.92	325,654,346.00	41898.3	73,404,251.66	33,175,152.79	3663.5	399,653,019.33	167,497,682.59	16359.5	47.418.974.29	18.206.344.08	2230.7	53,716,474.54	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 96.991.426.65 Federal aid $ 43.376.227.96 Miles 4172.4

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 6, NO. 10 ▽ DECEMBER, 1925

INTERCITY BUS LINE TERMINAL

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U. S. DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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THE MOTOR BUS AS A COMMON CARRIER

AN ANALYSIS OF BUS OPERATION IN EIGHT STATES

Reported by HENRY R. TRUMBOWER, Economist, United States Bureau of Public Roads

IN THE expanding use of the highways there is no more interesting or significant phase than the recent remarkable increase in the number of motor busses operating as common carriers. Highway engineers, with one eye constantly on the motor vehicle registration statistics, have learned by hard experience to make ample provision in their plans for increases in highway utilization which are nothing short of phenomenal. They have learned also the wisdom of a most respectful consideration of the demands of the motor truck; but this latest comer among the users of their highways will bear very particular watching.

Less than 10 years ago these public passenger vehicles of the highways were practically unheard of. To-day, in the eight States which provide the statistics for this article, they operate regularly over a mileage of the public highways equal to 79 per cent of the railroad trackage in the same States.

In the beginning they were mainly converted passenger cars and trucks, comfortless and uninviting. To-day, although the statistics of the eight States show that substantially a third of the total number in operation are five and seven-passenger touring cars, there is a large and constantly increasing number of commodious, comfortable, specially-designed vehicles which operate over the improved State roads with all the regularity and punctuality of the railroads.

The best of these vehicles have separate upholstered chairs for each passenger. There is no crowding. They run smoothly and swiftly on pneumatic tires. They are electrically lighted, glass inclosed, and heated. Riding in them in the most inclement weather is as comfortable as in a Pullman car.

Is this a passing fad, or is it but the beginning of a great transportation service of the future? To what extent will the busses replace privately-owned passenger automobiles? What are the practical limits of size and capacity? How numerous will they become? How will they affect the railroads? What are the demands they will make upon highway design and location? To these questions there seem at present to be no certain answers, except the first. The warmth of the public reception accorded the new types of busses seems to leave no doubt that they will continue in operation and increase in numbers.

In a search for the trends which will indicate the probable future development as a basis for future highway design we have analyzed the data obtainable from the files and records of the public service commissions of eight States, namely: Arizona, Connecticut, Kentucky, Maryland, New Hampshire, Oregon, Washington, and West Virginia.[1] As it was the purpose of the investigation to evaluate with respect to their use by the common carrier vehicles only the rural highways, the data analyzed relate wholly to the service offered between cities or between the cities and their suburbs. Data relating to strictly city operations have been excluded.

THE NUMBER AND LENGTH OF MOTOR BUS ROUTES

In the eight States there are 705 separate routes over which motor busses operate in suburban or interurban service. These routes have a total length of 18,196 miles and an average length per route of 25.8 miles. But though this was the average length of all lines in the eight States, the average length was found to vary considerably as between States. This is shown by Table 1, from which it will be seen that the variation runs from an average length of 9.7 miles in New Hampshire to 60.5 miles in Arizona. This table apparently shows also that the length of routes is likely to be relatively short in such States as Connecticut, Maryland, and New Hampshire where the population is dense and the distance between population centers small; and long in States like Oregon and Arizona where the population is sparse or the distances between centers great.

TABLE 1.—*Number and length of motor bus routes in the eight States*

State	Number of routes	Total length of routes	Average length of route
		Miles	*Miles*
Connecticut	53	924	17.4
New Hampshire	32	311	9.7
West Virginia	61	987	16.2
Kentucky	189	3,876	20.5
Arizona	39	2,358	60.5
Oregon	81	3,739	46.1
Washington	[1] 171	4,379	25.6
Maryland	79	1,622	20.6
Total	705	18,196	25.8

[1] The total number of routes in the State is 189. Mileage data are available for only 171.

Considering all routes in the eight States, however, it is evident from Table 2 that the short routes exceed all others. Routes 19 miles long or less constitute 51.6 per cent of the total number, and those less than 30 miles in length are almost three-quarters of the total. On the other hand, the routes 50 miles in length and over form only 11.9 per cent of the total number. It is obvious, so far as those States are concerned at least, that motor-bus operations are characterized largely by short-distance movements.

USE OF STATE HIGHWAYS AS BUS ROUTES

Excluding from the total mileage of the bus routes in the eight States 102 miles served by more than one route, the total mileage of highways used by the busses in these States is 18,094 miles, which constitute approximately 7 per cent of the total of 257,160 miles of highways in the same States, and 68 per cent of the mileage of the State highway systems, which, as shown by Table 3, is 26,610 miles.

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[1] The data were obtained directly from the records of the public service commissions in the States of Connecticut, Maryland, and New Hampshire. For the other States the material analyzed was obtained from operating summaries published in mimeographed form by the American Electric Railway Association, compiled therefor by the State utilities commission.

TABLE 2.—*Classification of motor-bus routes according to length*

Length of route (miles)	Number of routes									
	Connecticut	New Hampshire	West Virginia	Kentucky	Arizona	Oregon	Washington	Maryland	Total	Per cent
0–9	17	21	23	34	6	11	28	21	161	22.8
10–19	10	9	18	64	11	18	44	29	203	28.8
20–29	19		12	53	3	14	30	12	143	20.3
30–39	6	1	4	20		8	28	8	75	10.6
40–49	1		3	9	2	4	19	2	40	5.6
50–59				3	1	7	10	2	23	3.3
60–69		1		2	3	4	5	3	18	2.6
70–79			1	3	3	4	1	2	14	2.0
80–89					2	2	3		7	1.0
90–99				1	3		1		5	0.7
100 plus					5	9	2		16	2.3
Total	53	32	61	189	39	81	171	79	7 5	100.0

For the most part, the busses are operated only on the main highways joining centers of population, where sufficient traffic can be obtained. The feeder or secondary roads do not, in general, lend themselves to this type of common-carrier business. It will be seen, however, that a large percentage of the State highway systems, which include the more important roads, is already covered by motor-bus routes. In four of the eight States, i. e., Arizona, Maryland, Oregon, and Washington, more than half the State highway system mileage is so utilized. In Arizona and Washington, indeed, the motor busses operate over a mileage equivalent to that of the entire State system and a good many miles besides.

TABLE 3.—*Relation of motor-bus routes to highway mileage*

State	Motor-bus routes	Rural highways	Ratio of bus mileage to rural highway mileage	State highway system	Ratio of bus mileage to mileage of State highway system
	Miles	*Miles*	*Per cent*	*Miles*	*Per cent*
Connecticut	[1] 822	12,152	6.8	1,821	45
New Hampshire	311	13,841	2.2	1,367	23
West Virginia	987	35,173	2.8	3,594	27
Kentucky	3,876	68,704	5.6	8,000	48
Arizona	2,358	21,227	11.1	1,984	119
Oregon	3,739	45,475	8.2	4,464	84
Washington	4,379	45,816	9.6	3,133	140
Maryland	1,622	14,772	11.0	2,247	72
Total	18,094	257,160	7.0	26,610	68

[1] This is the net mileage of highways used by motor busses; lines having a total length of 102 miles use highways used by other lines.

Information with regard to the character of the roads used by the busses is available in the records of only two of the States, Connecticut and New Hampshire. In Connecticut 201 miles or 24.4 per cent of the total of 822 miles used are surfaced with concrete; 476 miles or 58 per cent with waterbound macadam; and 145 miles or 17.6 per cent with gravel. In New Hampshire the type of improvement is generally lower. Of what may be called high-type surfaces, including concrete, asphalt, and bituminous macadam, there are only 66 miles, which is less than 22 per cent of the 309 miles classified by type, and only 12 miles or 4 per cent are waterbound macadam; whereas 190 miles or 62 per cent are surfaced with gravel and 41 miles or 13 per cent are unsurfaced earth roads. The Maryland roads, although they are not classified in the records, are known to be of generally high type. As will appear later in detail, the average rates of fare in these three States are, for Connecticut, 4.6 cents per mile; for New Hampshire, 6.5 cents; and for Maryland, 4.5 cents per mile. To some degree, in all probability, the higher rate charged in New Hampshire may be due to the lower type of road improvement; but such a conclusion can not properly be drawn from the data available because of the differences which exist in the three States with respect to the length of routes and other factors.

COMPETITION OF BUSSES WITH RAILROADS

In the eight States in which 18,094 miles of highway are used for the regular operation of common carrier busses, there are only 22,994 miles of railroad. It is apparent, therefore, that, in point of territory served, the bus development is already almost as extensive as that of the railroads. Two States, in fact—Maryland and Oregon—have a mileage of bus routes which exceeds their rail mileage, as shown in Table 4. At the other extreme are the States of West Virginia and New Hampshire, in which the mileage of motor-bus routes is only about one-fourth of the railroad mileage; but, the mileage in these States being relatively small, the average ratio for the eight States is reduced only to 79 per cent.

TABLE 4.—*Motor-bus routes and railroad mileage*

State	Railroad mileage	Motor-bus route mileage	Ratio of motor-bus route mileage to railroad mileage
	Miles	*Miles*	*Per cent*
Connecticut	1,003	[1] 822	82
New Hampshire	1,239	311	25
West Virginia	4,057	987	24
Kentucky	3,955	3,876	98
Arizona	2,452	2,358	96
Oregon	3,347	3,739	111
Washington	5,494	4,379	80
Maryland	1,447	1,622	112
Total	22,994	18,094	

[1] This is the net mileage of highways used by motor busses; lines having a total length of 102 miles use highways used by other lines.

In view of the rapidly growing mileage of the bus routes considerable opposition has been made by steam-railroad operators on the ground that such operations come into direct competition with passenger railroad trains. It is therefore a matter of interest to see what part of the operation actually furnishes a service which competes with that of the railroads. For this purpose, in the States for which the data are available, the various bus routes have been divided into three classes according to the degree to which they enter into competition with the railroads.

Class I includes the routes which parallel the railroads and may therefore be said to come into direct competition with them. Class II includes routes which indirectly compete with railroads to the extent that their terminals are also connected in some manner by railroad, though to travel between the bus terminals by railroad would necessitate change of trains at one or more junctions and a roundabout journey. Finally, Class III includes the routes which, serving territory not served at all by railroad, are wholly noncompetitive. The mileage of the routes in each State which fall

Table 5.—*The competition of motor-bus routes with railroads*

State	Class I routes—Directly competitive				Class II routes—Indirectly competitive				Class III routes—Nomcompetitive				Total			
	Routes		Length		Routes		Length		Routes		Length		Routes		Length	
	Number	*Per cent*	*Miles*	*Per cent*	*Number*	*Per cent*	*Miles*	*Per cent*	*Number*	*Per cent*	*Miles*	*Per cent*	*Number*	*Per cent*	*Miles*	*Per cent*
Connecticut	23	43	463	50	17	32	311	34	13	25	150	16	53	100	924	100
New Hampshire	7	22	60	19	6	19	91	30	19	59	160	51	32	100	311	100
West Virginia	23	38	282	29	30	49	591	60	8	13	114	11	61	100	987	100
Kentucky	70	37	1,588	41	40	21	1,074	28	79	42	1,214	31	189	100	3,876	100
Arizona	10	26	728	31	14	36	1,061	45	15	38	569	24	39	100	2,358	100
Oregon	31	38	1,916	51	11	14	355	10	39	48	1,468	39	81	100	3,739	100
Washington	67	39	1,730	40	40	23	1,123	25	64	38	1,526	35	171	100	4,379	100
Total	231	37	6,767	41	158	25	4,606	28	237	38	5,201	31	626	100	16,574	100

within each of these classes is shown in Table 5, together with the percentage of the total in each class.

Considering all routes in seven of the States it will be noted that only slightly more than one-third of the number of routes and 41 per cent of the mileage come into direct competition with the railroads by paralleling rail lines. The routes which directly or indirectly compete with railroads are 62 per cent of the total number and their mileage is 69 per cent of the total mileage. Thirty-eight per cent of the number of routes and 31 per cent of the mileage extend between points one or both of which are not on any line of railroad and are therefore wholly noncompetitive. These 5,201 miles of noncompetitive bus lines constitute an addition to the common-carrier service provided by the railroads equal in mileage to approximately one-fourth of the total railroad mileage in the seven States.

As to the routes which are wholly noncompetitive with the railroads there can be no question, from the point of view of the public, of the propriety of their operation. Their use and the returns from the use will determine in each case whether the operation is feasible from the point of view of the operator. As to the routes of Class II which indirectly compete with the railroads by connecting terminals that are also connected by railroads there may be some ground for question as to the expediency of the operation; but with respect to the routes actually operated it is fair to presume that this question has been investigated and decided on its merits favorably to the bus operators by the public service commission in each case. It is worthy of note that these two classes of routes which either compete not at all or only indirectly with the railroads include 63 per cent of the total number of routes in the eight States, and that their combined mileage is 59 per cent of the total mileage.

In Kentucky there are still 15 counties which do not have any railroad lines. These particular counties are served by 19 motor-bus routes over which 31 busses are operated.

THE NEW HAMPSHIRE ABANDONMENTS OF BOSTON & MAINE NOT CAUSED BY BUS COMPETITION

Even where the bus line directly parallels a railroad it does not follow from that fact alone that the bus service is not a desirable public convenience. With respect to such bus lines actually in operation there is, indeed, weighty evidence that the service is really desirable in the facts that the operations have been permitted by the public service commissions and that they are able to continue in competition with the railroads.

Considerable interest has been shown in the announced policy of the Boston & Maine Railroad Co. to abandon 1,000 miles of its track. It has been inferred that competition by motor vehicles and motor busses may be partly to blame for the reduced traffic which appears to necessitate the elimination of certain of the rail lines. As a matter of fact, it appears that as far as bus lines are concerned the competition with this railroad is very limited.

Ninety-six miles of the trackage which the railroad desired to abandon is in the State of New Hampshire, one of the eight States under consideration in this article. A special effort has been made, therefore, to ascertain to what extent the abandonment of these 96 miles may have become desirable as a result of motor-bus competition. It is found that none of the mileage proposed for abandonment has to meet any motor-bus competition, either direct or indirect. It can not be claimed, therefore, that motor busses have taken away any of the passenger traffic which these lines formerly enjoyed.

As a matter of fact, only a very small portion of the total railroad mileage in New Hampshire has any motor-bus competition. The total railroad mileage in the State is 1,239 miles; the motor-bus routes which run parallel amount to only 60 miles, or 4.9 per cent. If the routes of Class II are also included as being to a certain extent competitive, we find that there are still only 151 miles of competing bus routes, which is only 12.2 per cent of the total railroad mileage.

RELATIVE SERVICE OF COMPETING BUS AND RAIL LINES

As almost invariably the rates charged by the bus operators are higher than the rates of competing railroads, the fact that the bus lines are able to continue in business must be due to some superiority of the service rendered.

With respect to the routes of Class II, this superiority is manifest in more direct routes, lower total charges for service between the terminals, and more frequent service than the railroads are able to offer. Thus the total mileage of the Class II bus lines in Connecticut is 297 miles. The sum of the distances one would have to travel by railroad in going from one terminal to the other of these bus routes would be 610 miles, or a little over twice the length of the bus routes. The total of the motor-bus fares for the travel of 297 miles would be $14.20, and the railroad fares for the journeys between the same terminal points would total $21.96, which is 54 per cent greater.

In many cases the Connecticut Class II bus lines materially shorten the distance between two points which are connected by railroad only in a very roundabout way. The most extreme case found is that of the bus route between Torrington and Bantam, over which the distance is 10 miles and the fare 60 cents. Travel between these two points by railroad involves a total distance of 73 miles, with two train changes, at a cost of $2.63. Another example from Connecticut is the trip from Canton to Collinsville, which is only 2 miles by motor bus and 12 miles by railroad. From Colchester to New London is 20 miles by motor bus, with a fare of $1; the train between the same two points makes a trip of 66 miles with three changes for a fare of $2.38.

Where the bus routes parallel the rail lines it will generally be found that the ability of the bus operators to meet the railroad competition is due to the greater frequency of service. Where the time required for the trip and the fares can be compared, the advantages lie wholly with the railroads. Considering all Class I bus routes in Connecticut, the total time consumed in going from terminal to terminal of all the routes amounts to 21 hours and 55 minutes. To make the same trips by railroad passenger trains takes only 18 hours and 40 minutes. The time of the motor-bus journeys exceeds that of the trains by 18 per cent. The total railroad fares would be $17 and the motor-bus fares $20.35, or 20 per cent more. The ability of the motor bus to get business under such conditions must be due, therefore, to the more frequent operation and to the more complete service they render by picking up and discharging passengers in the central business districts of the cities through which they pass.

As to the greater frequency of the motor-bus service there is direct evidence in the Connecticut data, which shows that the busses of Class I make 314 trips every day between points served by the railroads with only 190 trips. Each motor-bus route averages 13.7 trips per day; whereas the parallel steam roads average only 8.7 daily trips. According to this, the bus service is 50 per cent more frequent.

In granting permits to operators of motor busses the public service commissions of the several States are inclined to give protection to railroad service where it appears that public necessity does not require the additional service and that the proposed motor-bus service would cut into the much-needed revenues of the rail carrier. Where the railroad service is reasonably frequent and where there is not enough traffic to support more than one mode of transportation, the commissions are generally inclined to oppose the inauguration of the new service. It is the general practice not to allow competition between common carriers; this holds true also where a second motor-bus operator proposes to give service over a route already served by a bus line and where it appears that there is not enough traffic in sight to support the additional service.

ATTITUDE OF THE PUBLIC SERVICE COMMISSIONS ILLUSTRATED

The attitude of the public service commissions is illustrated by the findings of the Connecticut commission in the matter of an application to establish a motor-bus line between the cities of Hartford and Meriden. The distance between the two cities is 18 miles. The New York, New Haven & Hartford Railroad Co., operates 19 trains each way on week days between the two points. The railroad fare is 65 cents and the running time 30 minutes.

It was proposed to operate busses on the highway connecting the two cities, giving hourly service at the same fare charged by the railroad, though the running time was to be 50 minutes. The evidence presented at the time of the hearing disclosed that the railroad carried approximately 300 single-trip passengers daily and 200 commuters. The commuter's fare is about 20 cents. It was pointed out by the commission that the motor-bus line could not take care of this commuter traffic, nor could it meet this low rate of fare. The result of the motor-bus operation would be to take away from the railroad a considerable number of the other passengers and thereby deprive the company of a portion of the revenues produced by the regular fare.

It was feared by the commission that such circumstances would react to the detriment of the commuters favored by the low rates by forcing an increase in their fare and probably leading to a reduction of service. According to the testimony as to public convenience and necessity, it was shown that the only persons urging the establishment of the bus service were those individuals residing along the trunk highway connecting Hartford and Meriden, and not enough prospective traffic was in sight from those sections without the through traffic to warrant the operation. The application was therefore dismissed and the certificate denied.

An anlaysis of the location of the motor-bus routes in Connecticut shows that there are no bus routes running parallel to steam rail lines over which frequent and well-spaced train service is afforded and where the bus service would cater largely to passenger traffic moving between large centers of population. There are, for instance, no bus lines operating on the Boston Post Road from the New York line to New Haven, except the line operated by the street railway company between New Haven and Bridgeport. There is none between New Haven and Hartford, or between Hartford and Waterbury. In each case it appears that there is frequent and ample steam rail service.

Now and then the commission faces a situation which involves two or more applications on the part of prospective bus operators desiring to give service over a route which the commission is ready to approve. It has then to decide which one of the candidates for a certificate should be favored. In deciding such cases the commission bears in mind the financial responsibilities of the respective applicants, their past records and performance, the general character of equipment and service proposed, and the expressed preference of the people of the communities to be served.

MOTOR-BUS FARES

In only three of the eight States was it possible to obtain information relative to motor-bus fares. In Connecticut and Maryland the average rates are approximately 4.5 cents per mile; in New Hampshire, 6.5 cents per mile. As the railroad fare is 3.6 cents per mile, it is clear that people do not ride in motor busses to escape payment of unreasonably high railroad fares; for not only are the regular railroad rates considerably less than the average bus fare, but the former offer to regular riders commutation rates which make the disparity between the average rail rates and bus rates even greater.

The fares charged by all New Hampshire bus lines and by representative lines in Connecticut and Maryland are classified in Table 6. The lowest fare charged by any line in the three States is the rate of 2 cents per mile charged by one line 9.8 miles in length in Maryland. The highest rate is the charge of 18.8 cents per mile on an 8-mile route in New Hampshire. The fares in this State are generally higher than in the other two States and, as shown by Table 6, there are 14 of the routes in the State, or 44 per cent of the total number, that charge higher rates than any charged in either of the other States. The 15-cent rate is charged on a 3-mile route from Sunapee to Sunapee Station, where 45 cents is charged on a 10-passenger bus.

Table 6.—*Fares charged by interurban and suburban bus lines in Connecticut, New Hampshire, and Maryland*

Rate of fare per mile (cents)	Connecticut routes		New Hampshire routes		Maryland routes	
	Number	*Per cent*	*Number*	*Per cent*	*Number*	*Per cent*
2 to 3	2	4.2	1	3.1	1	3.1
3 to 4	7	14.9	4	12.5	6	18.8
4 to 5	21	44.7	3	9.4	11	34.4
5 to 6	11	23.4	7	21.9	13	40.6
6 to 7	3	6.4	2	6.3	1	3.1
7 to 8	3	6.4	1	3.1		
8 to 9			1	3.1		
9 to 10			4	12.5		
10 to 11			2	6.3		
12 to 13			5	15.6		
15 to 16			1	3.1		
18 to 19			1	3.1		
Total	47	100.0	32	100.0	32	100.0

The lowest rate charged in Connecticut is 2.4 cents per mile, the highest 7.7 cents. The lowest in New Hampshire is 2.5 cents and the highest the 18.8-cent rate referred to above. The lowest in Maryland is the 2-cent rate already mentioned, and the highest rate charged in this State is 6 cents a mile. In Connecticut only 7 out of the 47 routes for which the fare information is available, or 15 per cent, charge fares equal to or less than the standard railroad rate of 3.6 cents per mile. In Maryland 4 of the 32 routes, or 13 per cent, have similarly favorable rates, and approximately the same relative situation exists in New Hampshire.

Although it is evident from the above that the bus lines are not able to meet the railroad rates, a comparison of the rates charged on routes in direct competition with railroads with those charged by the routes which do not have such direct competition indicates in each of the States that the railroad competition does operate to keep the bus fares at a lower level. In Connecticut, for example, the average rate of fare charged by the bus lines which operate on routes that parallel steam railroads is 4.3 cents a mile; on the lines not directly in competition with railroads it is 4.9 cents. In New Hampshire the Class I lines charge an average fare of 4.4 cents a mile, those in Class II charge an average of 5.5 cents, and the Class III lines, which have no rail competition either direct or indirect, charge an average fare of 7.9 cents. It is evident that the actual or potential competition of railroad lines tends to keep the bus fares at a lower level than where such competition is not present.

LICENSE FEES AND TAXES VARIABLE

The licenses, special permit fees, and gasoline taxes levied on motor-bus operations vary considerably in the States under consideration. In order to make a comparison, the following specifications have been taken as a basis for the computation of fees: 20-passenger bus, pneumatic tires, 9,000 pounds weight, 30 horsepower, price $8,000, 50,000 miles travel annually, $12,000 gross annual receipts, and 7,142 gallons annual gasoline consumption. Such a vehicle would pay in each of the States the fees and taxes shown in Table 7.

Table 7.—*Motor-bus license fees and taxes*

State	Annual license fee	Gasoline tax	Total	Personal-property tax in addition
Connecticut	$82.50	$71.42	$153.92	Yes.
New Hampshire	276.00	142.84	418.84	Yes.
West Virginia	666.67	249.97	916.64	Yes.
Kentucky	270.00	214.26	484.26	Yes.
Arizona	25.00	214.26	239.26	No data.
Oregon	177.00	214.26	391.26	No.
Washington	250.00	142.84	392.84	Yes.
Maryland	1,428.57	142.84	1,571.41	Yes.
Average	396.96	174.09	571.05	

As in six of the seven States for which information is available personal-property taxes are levied upon the busses in addition to the license and gasoline taxes shown in Table 7, these taxes must be considered, in these six States at least, as wholly of the nature of special taxes for the use of the road and comparable directly with the railroad expenditures for maintenance of way. As will be observed, the lowest taxes are levied in Connecticut, the highest in Maryland, and the total tax in Maryland for the assumed vehicle is more than ten times the Connecticut tax. The average tax, $571.05 for the eight States, is 4.75 per cent of the assumed gross annual receipts of $12,000.

Under the Maryland law motor-bus operators are required to file reports covering their financial operations for the year. Although much of the data contained in these reports are not, in most cases, presented in accordance with good accounting principles, the items of gross revenues, taxes and licenses, and gasoline and oil expenditures are, in all probability, fairly reported. In their annual reports for 1924, 13 motor-bus operators reported a tax and license expenditure of $19,932, the same operators showing gross receipts of $344,587. The taxes and license fees of these operators, therefore, amounted to 5.8 per cent of the total revenues. The range was from 3.7 to 8.2 per cent, but only four exceeded the average.

The largest operator, with 50 busses, had a revenue of $141,709; the smallest took in but $1,128 with his Ford touring car which he used as a passenger and mail carrier. Fourteen operators with gross receipts of $367,934 expended for gasoline and oil, according to their reports, $52,704, or 14.3 per cent of their total revenues. The ratio of gas and oil expenditures to revenues ranged from 8.6 to 30.5 per cent. The latter figure, however, was unusual, and the next below it was 20 per cent. It would be a fair generalization to say that the gasoline and oil costs were from 10 to 20 per cent of the passenger receipts.

THE CAPACITY OF BUSSES IN USE

Over a third of the common carriers classified in this analysis as busses are five and seven passenger touring cars. In fact, three of the vehicles so classed in Kentucky have accommodations for less than five passengers, the smallest being a one-passenger vehicle.

The type of vehicle generally termed a "bus" is first encountered in the 8 to 14 passenger group. The vehicles of this type, carrying eight passengers and more number, in the seven States for which the information is obtainable, 923 and comprise 64.6 per cent of the total number of common carriers in these States. The large busses of 25 passengers capacity and over constitute only 5.1 per cent of the total number.

A summary of the capacity of the busses in operation is presented in Table 8, from which it will be seen that the 1 to 7 passenger group comprises 35.4 per cent of the total number; the 8 to 14 passenger group 25.3 per cent; the 15 to 24 passenger group 34.2 per cent; and the 25 passenger class 5.1 per cent as stated above.

Table 8.—*Classification of interurban and suburban motor busses according to capacity*

State	Number of busses of various capacities				
	1 to 7 passengers	8 to 14 passengers	15 to 24 passengers	25 passengers and over	Total
Connecticut	40	30	77	21	168
New Hampshire	12	18	14		44
West Virginia	91	14	43	1	149
Kentucky	148	70	51	1	270
Arizona	62	10	17	3	92
Oregon [1]					
Washington	147	173	191	35	546
Maryland	7	47	96	11	161
Total	507	362	489	72	1,430

[1] Capacity data lacking.

The 168 busses operating on interurban routes in Connecticut have an average capacity of 16.3 passengers. In this State there is apparently no marked tendency to confine certain types of vehicles to the short or long routes. The 20-passenger bus is found on both the short and long routes. The seven-passenger car is also found on the various routes up to 30 miles in length. It is evident in this State that the capacity of the bus is adjusted to the demands of traffic regardless of the length of the route.

In New Hampshire the average seating capacity of the busses is 12.8 passengers; and the most popular sizes are the 5, 14, and 20-passenger vehicles of which there are, respectively, 9, 8, and 7 vehicles each among the total of 44 busses operating in the State.

In West Virginia the average capacity is only 9.9 passengers and the predominant sizes among the 149 busses in operation are the 5, 7, and 16-passenger types, of which there are, respectively, 22, 69, and 22 busses. Over 61 per cent of the busses in this State are five and seven passenger touring cars.

Kentucky which, next to Washington, has the largest number of busses in operation, has, next to Arizona, the smallest average seating capacity per bus. The average is 9.7 passengers. Over half the so-called busses are of seven-passenger capacity or less, and a third are of small capacity, ranging from 8 to 15 passengers each. The larger busses, seating from 16 to 26 passengers, comprise only 12 per cent of the total number.

Arizona, in which the average capacity of the busses is only 9.6 passengers, has a larger percentage of the touring-car busses than any other State analyzed. Only 32.3 per cent of the vehicles in this State are of the strictly bus type.

CLASS III ROUTES IN OREGON HAVE SMALL CAPACITY

Although a complete classification of the busses in Oregon according to capacity is not possible, the data show that the average seating capacity of the 310 busses in operation is 12.9 passengers. It follows, therefore, that a large number of the busses in operation must be of the larger sizes. In this State the records show that the busses operating on Class I routes, which parallel rail lines, have an average seating capacity of 16 passengers; those operating on the Class II routes, indirectly in competition with railroads, have an average seating capacity of 11.5 passengers; and those operating on Class III routes, which do not compete at all with the railroads, seat an average of only 8.1 passengers. It is apparent that the carriers in this State operating on routes leading off from the railroads are of relatively small capacity. Although the Class III routes constitute 48.2 per cent of all routes, the capacity of the busses operating on these routes is only 20 per cent of the total bus capacity on all routes in the State.

There are 546 busses in operation in Washington, more than in any of the other States, and these busses have a total seating capacity of 7,771 passengers, equivalent to that of 111 railroad passenger coaches with seats for 70 passengers each. The average capacity of the Washington busses is 14.2 passengers. Although the seven-passenger touring car, of which there are 116, is used more than any other size, 53 per cent of the vehicles fall within the capacity group of 10 to 19 passengers. Eighty-two per cent have a passenger capacity of less than 20.

A comparison of the Maryland records for 1924 and 1925 shows that, although the number of busses in operation increased from 106 to 161, the relative number of vehicles of the several capacity classes did not change materially. The classification in Table 9 shows that the 8 to 14 passenger group increased both in number and percentage more than any other group. The only other group to increase in percentage was the 21 to 28 passenger group; both the 5 to 7 and the 15 to 20 passenger vehicles falling off in percentage although they increased in numbers. The increase in the number of large-capacity busses, those of 20 passengers and over, is partially accounted for by the inauguration of bus service between Baltimore and Washington and between several other large cities.

Table 9.—*The capacity of Maryland busses in 1924 and 1925*

Capacity	Busses operating in 1924		Busses operating in 1925	
	Number	*Per cent*	*Number*	*Per cent*
5–7 passengers	5	4.7	7	4.3
8–14 passengers	26	24.5	47	29.2
15–20 passengers	53	50.0	70	43.5
21–28 passengers	22	20.8	37	23.0
Total	106	100.0	161	100.0

INTERSTATE BUS OPERATION

Of the 53 interurban motor-bus routes authorized by the Public Utilities Commission of Connecticut, 10 routes, or 19 per cent, are interstate in character. Two routes extend into the State of New York, five extend into Massachusetts, and three into Rhode Island. It is reported that there are other lines engaged in interstate operation, the operators of which deny the jurisdiction of the Connecticut commission on the ground that they carry passengers in interstate commerce only.

In recent decisions of the United States Supreme Court in the cases of Bush *v.* Public Service Commission of Maryland, No. 185, Mar. 2, 1925, and Bush *v.* Kuykendall, No. 345, Mar. 2, 1925, the court held in effect that the State public service commissions may not prohibit the operation of busses engaged exclusively in interstate commerce by withholding a certificate of public convenience and necessity; although the court held that the inhibition in this respect does not prevent the States from demanding full compliance with all State laws and regulations with respect to the use of the road designed to preserve the roads and promote the safety of road users.

Nine of the 61 routes in West Virginia, or nearly 15 per cent, are interstate routes; 14 of the 189 routes in Kentucky, or 7.4 per cent, are similarly classed; and 7, or 3.7 per cent, of the 189 routes in Washington are also interstate. The exact number of interstate routes in the other States is not avialable. There are several in Maryland, the most important of which are the two operating between Baltimore and Washington in the District of Columbia. In Arizona there is one route 340 miles long, which extends from Phoenix to Riverside, Calif.; and in Oregon the longest route in the State is one which extends for 346 miles from Portland south to the Oregon-California line and by its connections with California lines provides what amounts to an interstate service.

DAILY TRIPS AND MILEAGE

The available records of Connecticut and New Hampshire provide information with regard to the number of trips made each day by busses operating on routes of various lengths and the bus mileage on the various routes. This information is tabulated in Tables 10, 11, and 12.

TABLE 10.—*Daily trips on bus routes of various length in Connecticut and New Hampshire*

Length of route (miles)	Connecticut		New Hampshire	
	Number of daily trips	Per cent	Number of daily trips	Per cent
0–5	78	11.6	182	64.5
5–10	218	32.4	48	17.1
10–15	62	9.2	44	15.6
15–20	26	3.9	4	1.4
20–25	148	22.0		
25–30	72	10.8		
30–35	48	7.1	2	.7
35–40	16	2.4		
45–50	4	.6		
60–65			2	.7
Total	672	100.0	282	100.0

TABLE 11.—*Classification of Connecticut routes according to daily bus mileage*

Daily bus mileage	Routes	
	Number	*Per cent*
0–100	14	26
100–200	23	44
200–300	6	11
300–400	6	11
400–500	2	4
800–900	2	4

TABLE 12.—*Daily bus mileage on routes of various lengths in New Hampshire*

Length of route	Daily bus mileage	
Miles	*Miles*	*Per cent*
0–5	582	34
5–10	318	19
10–15	536	32
15–20	76	4
30–35	64	4
60–65	120	7
Total	1,696	100

The largest numbers of daily trips in both States are made on routes of 10 miles or less in length. In Connecticut 51 per cent of all routes are less than 20 miles long and the trips made over these routes are 57 per cent of the total number of daily trips made in the State. In New Hampshire, where the routes are even shorter than in Connecticut, 64.5 per cent of the total number of daily trips are made on routes less than 5 miles in length.

In Connecticut the daily bus mileage on 70 per cent of the routes is less than 200 miles, and 26 per cent have a daily mileage of less than 100. Two routes—those between New Haven and Waterbury and Bridgeport and Waterbury—by reason of the frequency of the service have an extraordinarily large daily mileage. On the New Haven-Waterbury route, 24 miles in length, there are 34 single trips a day and a daily operation of 816 bus-miles. On the Bridgeport-Waterbury route, 30 miles long, there are 28 single trips daily resulting in a daily operation of 840 bus-miles. The published time schedules of the various operators in this State show that additional trips are made over many of the routes on Saturdays, Sundays, and holidays. It may safely be said that the bus mileage on these days is from 10 to 20 per cent higher than on ordinary week days.

INDIVIDUALS AND PARTNERSHIPS CONTROL MAJORITY OF ROUTES

The operation of the majority of the routes in most of the States is in the hands of individuals or partnerships, most of which operate only one or two vehicles. In all States, however, corporations have entered the field, and, as would be expected, these organizations operate a greater number of vehicles. In one of the States, Maryland, there is a corporation which operates 50 or more busses over a number of routes, but that is exceptional. The certificate of convenience and necessity required by law protects the small operator so long as he serves the transportation needs of the public in a satisfactory and adequate manner, and tends to preserve the small-scale operation.

Sixty-three per cent of the operators in Connecticut are individuals who control 61 per cent of the total number of routes, but operate only 34 per cent of the total number of busses. Contrasted with this, the corporations in the State, which constitute only 29 per cent of the number of operators, control 31 per cent of the routes and operate 60 per cent of the busses. Partnerships in this State constitute 8 per cent of the number of operators, control 8 per cent of the routes, and operate 6 per cent of the busses.

(Continued on page 232)

EFFICIENCY IN CONCRETE ROAD CONSTRUCTION

A REPORT OF OBSERVATIONS MADE ON GOING PROJECTS BY THE DIVISION OF CONTROL, BUREAU OF PUBLIC ROADS

Reported by **J. L. HARRISON**, Highway Engineer

PART II.—THE TRANSPORTATION OF MATERIALS

IN THE first article of this series, special attention was given to the causes generating delay at the mixer because the mixer is the pacemaker for the job. Slow operation of the mixer must be corrected at the mixer, but other causes of delay point back into the underlying organization, and their correction involves a study of the other parts of the organization and their relation to each other. A concrete paving organization is composed of a number of units which, for efficiency, must be synchronized. If the mixer is to turn out 1,000 feet of paving every day, the subgrade gang must prepare that amount of subgrade every day, the form setters must set 1,000 feet of forms, the railroad must deliver the materials for that quantity of concrete every day, the batcher plant must handle it, the trucks must haul it, the finisher must finish it, and the water supply must be adequate to mix it and to cure it. There must be a smooth consistency of progress in all of these operations—no running ahead, no lagging behind. In short, every part of the construction machine must be in balance with every other part—each maintaining an even, steady pace and all gauged to fit the pacemaker, the mixer.

But when the causes of low production are measured at the mixer and analyzed as in the first of this series of articles, it generally develops that some, at least, of the delays, though expressed in such simple terms as truck shortage, inadequate water supply, waiting for subgrade, etc., point to basic and often far-reaching lack of balance in the organization, the result of failure to appreciate that a concrete construction organization is really composed of a number of smaller organizations each of which, in size and equipment, must be gauged by the production it is desired to obtain from the mixer organization. This is outstandingly true of the transportation, which is not surprising, for the problem of balancing the delivery of materials to the mixer so as to maintain full-capacity output is not always a simple one, and every new job offers a new transportation problem.

Aside from central mixing plant operation which is not touched upon in this series of articles, the three recognized systems of material delivery are:

1. Delivery by batch trucks from the material yard to the mixer, each batch being dumped directly into the skip.
2. Delivery by industrial railway from the material yard to the mixer, each batch in a separate batch box which, at the mixer, is lifted from the cars by a crane so that its contents may be deposited either in the skip or in an elevated hopper which is a part of the mixer.
3. Delivery by wagons or trucks to the job where the aggregate is deposited in piles along the subgrade and moved to the skip in wheelbarrows.

The use of batch trucks is the method most commonly employed, and the trucks are generally either of single-batch or two-batch capacity. Larger trucks are sometimes seen but are not in common use. The delivery problem to-day is, therefore, largely a truck problem, involving, in the first instance, a choice between single-batch and two-batch trucks.

The two-batch trucks are generally either of the heavy-duty or high-speed type. Those of single-batch capacity are almost exclusively of the planetary-transmission type and, with the modifications in transmission now very generally made in the interest of service under excessive load, and a dump body satisfactory for this sort of work, they cost the contractor about $750 each. Heavy-duty and high-speed trucks, equipped to carry two batches, cost from $4,000 to $5,000. The heavy-duty, two-batch truck is generally rated for service at an average speed of about 12 miles an hour; the single-batch truck at an average speed of 15 miles an hour; and the high-speed truck at an average speed of from 18 to 20 miles an hour. These speeds may also be expressed as 5 minutes per mile for heavy-duty trucks, 4 minutes per mile for single-batch trucks, and 3 minutes per mile for high-speed, two-batch trucks, the time for a round trip (1 mile out and 1 mile back) being twice that for 1 mile. Field studies show that actual operating speeds agree, in a general way, with these rated speeds. Occasionally an effort is made to run trucks faster than the speed for which they are designed. This increases the loads delivered per hour but wears out the trucks, increasing depreciation as well as lost time awaiting repairs and the cost of the repairs.

ESTIMATING THE NUMBER OF TRUCKS REQUIRED

The field studies show that in the operation of a single-batch truck four minutes are needed to cover servicing at a well-equipped material plant and at the mixer while for a two-batch truck six minutes are required. These servicing operations are: (1) Loading sand and coarse aggregate, (2) loading cement, (3) turning on the turntable, and (4) discharging at the mixer. To the time required for these operations there must generally be added about one minute for unavoidable delays, and the total time as given above includes this allowance.

There result, then, the three following formulas which may be used in determining the time required per trip for each of the standard types of trucks:

For heavy-duty, two-batch trucks, $T=10d+6$ . . . . . . (1)

For ordinary single-batch trucks, $T=8d+4$ . . . . (2)

For high-speed, two-batch trucks, $T=6d+6$ . . . . (3)

in which T is the time in minutes required for a round trip, and d is the distance in miles from the batcher plant to the mixer.

For example, if the haul is 2.6 miles, how many trucks of each of the above types would be necessary in order to provide a full supply of material for a mixer, operating at 100 per cent efficiency, the working day being 10 hours?

Solution: Batches required, 48 batches per hour for 10 hours = 480 batches. Working day, 10 times 60 = 600 minutes.

TABLE 1.—*Stop-watch record of truck operation under very lax supervision*

Time: August 21, 1925. Number of single-batch trucks: 20. Condition of route: Good except for some soft spots. Distance loading bins to cement shed: 300 feet. Trucks turn through forms 150 feet from mixer. Haul distance plant to mixer: 1.1 miles.

Truck No.	Delay at plant	Time required to load cement and aggregates	Travel time loaded	Speed loaded	Delay at site of paving	Turning and dumping time	Travel time empty	Speed empty	Total time per round trip
	Min. Sec.	*Min. Sec.*	*Min. Sec.*	*Mi. p. hr.*	*Min. Sec.*	*Min. Sec.*	*Min. Sec.*	*Mi. p. hr.*	*Min. Sec.*
6	1 26	1 34	9 30	10.1	0 30	1 30	7 00	13.7	21 30
16	1 22	1 38	9 00	10.7	1 00	1 30	6 30	14.8	21 00
13	2 21	1 39	9 00	10.7	1 00	1 30	7 30	12.8	23 00
20	3 23	1 37	8 00	12.0	1 30	1 30	7 00	13.7	23 00
7	4 04	1 26	7 30	12.8	1 30	1 30	6 00	16.0	22 00
27	3 21	2 09	8 30	11.3	1 00	1 30	7 30	12.8	24 00
11	3 04	1 26	8 30	11.3	3 00	1 30	5 30	17.4	23 00
28	2 19	1 41	11 30	8.4	2 00	1 30	25 00	3.8	44 00
3	2 52	1 38	7 30	12.8	2 00	1 30	6 30	14.8	22 00
12	3 22	1 38	8 00	12.0	1 30	1 30	9 30	10.1	25 30
25	2 35	1 55	12 30	7.7	2 00	1 30	7 30	12.8	28 00
5	3 24	1 36	8 00	12.0	2 00	1 30	7 30	12.8	24 00
17	1 54	2 06	7 30	12.8	2 30	1 30	6 00	16.0	21 30
23	3 10	1 20	8 00	12.0	2 00	1 30	6 30	14.8	22 30
6	1 27	1 33	8 00	12.0	0 30	1 30	7 00	13.7	20 00
1	1 49	1 41	7 30	12.8	1 30	1 30	6 00	16.0	20 00
19	2 01	1 29	7 30	12.8	0 30	1 30	9 30	10.1	22 30
15	1 40	1 20	9 00	10.7	0 30	1 30	7 30	12.8	21 30
13	1 09	2 21	23 00	4.2	3 30	1 30	8 00	12.0	39 30
7	2 59	1 31	6 00	16.0	0 00	1 30	5 00	19.2	17 00
20	3 09	2 21	11 30	8.4	6 30	1 30	8 00	12.0	33 00
27	2 16	1 44	8 00	12.0	0 00	1 30	8 30	11.3	22 00
11	1 31	2 59	7 30	12.8	6 30	1 30	7 00	13.7	27 00
3	0 48	3 42	7 30	12.8	6 00	1 30	7 00	13.7	26 30
12	1 27	1 33	11 30	8.4	1 30	1 30	29 00	3.3	46 30
23	2 15	1 45	9 00	10.7	2 30	1 30	8 00	12.0	25 00
25	1 42	1 48	11 30	8.4	4 00	1 30	8 30	11.3	29 00
6	2 33	1 27	6 30	14.8	3 00	1 30	8 00	12.0	23 00
16	2 16	1 44	7 00	13.7	3 30	1 30	7 30	12.8	23 30
7	1 56	1 34	7 00	13.7	4 00	1 30	5 30	17.4	21 30
2	1 51	2 09	11 00	8.7	1 00	1 30	11 30	8.4	29 00
Average	2 18	1 48	9 06	10.5	2 12	1 30	8 37	11.1	25 31

NOTE.—The slowness of the trucks is in part due to the subgrade but much more to the fact that they are all individually owned. The drivers are responsible for their repairs and the lack of a central repair shop for trucks and the general disorganization due to the trucks not being under one management is reflected in the greater time lost and the greater time required for the various operations.

TABLE 2.—*Stop-watch record of truck operation under efficient supervision*

Time: July 31, 1925. Number of single-batch trucks: 12. Condition of route: Good. Distance turntable to mixer: 200 feet. Distance batcher to cement shed: 500 feet. Haul distance plant to mixer: 1,1 miles.

Truck No.	Delay at plant	Time required to load sand and gravel	Time required to load, dump, and cover cement	Travel time loaded	Speed loaded	Delay at site of paving	Time required to turn on turntable	Time required to dump load	Travel time empty	Speed empty	Total time per round trip
	Min. sec.	*Min. sec.*	*Min. sec.*	*Min. sec.*	*Mi. p. hr.*	*Min. sec.*	*Min. sec.*	*Min. sec.*	*Min. sec.*	*Mi. p. hr.*	*Min. sec.*
17	0 00	0 16	0 35	5 00	13.2	2 30	0 15	0 10	5 00	13.2	13 46
30	1 00	0 16	0 35	5 00	13.2	2 30	0 20	0 09	5 00	13.2	14 50
19	1 00	0 11	0 42	6 00	11.0	1 00	0 30	0 10	5 00	13.2	14 33
18D	0 00	0 11	0 41	4 00	16.5	3 00	0 15	0 07	4 00	16.5	12 14
10	0 30	0 20	0 53	4 00	16.5	3 00	0 30	0 10	4 00	16.5	13 23
15	1 00	0 10	0 47	5 00	13.2	2 00	0 28	0 06	4 00	16.5	13 31
4	1 00	0 20	0 37	5 30	12.0	2 30	0 21	0 06	4 30	14.7	14 54
14	2 00	0 14	0 32	5 00	13.2	2 00	0 23	0 08	4 30	14.7	14 47
17D	2 00	0 14	0 51	4 00	16.5	2 30	0 18	0 08	4 00	16.5	14 01
8	1 00	0 16	0 40	4 00	16.5	2 30	0 17	0 09	5 00	13.2	13 52
7	0 30	0 20	1 00	4 00	16.5	2 00	0 19	0 09	5 30	12.0	13 48
32	0 30	0 11	0 55	5 00	13.2	2 30	0 26	0 07	4 00	16.5	13 39
17	0 00	0 26	0 45	5 00	13.2	2 00	0 24	0 11	4 00	16.5	12 46
30	0 00	0 11	0 45	5 00	13.2	3 00	0 32	0 07	4 00	16.5	13 35
19	0 30	0 14	0 52	4 30	14.7	2 30	0 18	0 11	5 30	12.0	14 35
18D	1 00	0 12	0 37	4 00	16.5	3 00	0 26	0 07	5 00	13.2	14 21
10	0 00	0 14	0 42	5 00	13.2	2 00	0 22	0 09	5 00	13.2	13 27
15	0 00	0 17	0 55	4 30	14.7	4 00	0 26	0 07	4 00	16.5	14 15
4	0 00	0 18	0 38	4 30	14.7	1 00	0 28	0 11	4 00	16.5	11 05
14	1 00	0 14	0 44	4 00	16.5	1 30	0 26	0 09	4 30	14.7	12 33
17D	1 00	0 13	0 51	3 30	18.8	3 00	0 25	0 11	4 00	16.5	13 10
12	0 00	0 10	0 48	5 30	12.0	0 30	0 20	0 09	5 00	13.2	12 27
8	0 00	0 16	0 42	5 00	13.3	0 30	0 31	0 11	5 00	13.2	12 10
32	1 30	0 16	0 56	4 00	16.5	1 00	0 17	0 08	4 30	14.7	12 37
7	0 30	0 15	0 48	4 00	16.5	2 00	0 25	0 11	4 30	14.7	12 39
17	0 30	0 13	0 51	5 30	12.0	0 00	0 24	0 13	5 00	13.2	12 41
30	0 00	0 10	0 53	5 00	13.2	0 30	0 24	0 08	5 00	13.2	12 05
19	1 00	0 14	0 42	5 00	13.2	1 00	0 26	0 11	4 00	16.5	12 33
18D	2 00	0 20	0 36	4 00	16.5	1 00	0 29	0 09	4 00	16.5	12 34
10	2 00	0 16	0 45	4 00	16.5	1 00	0 32	0 11	5 00	13.2	13 41
15	2 00	0 18	0 51	4 00	16.5	2 00	0 31	0 10	4 00	16.5	13 50
Average	0 45	0 15	0 45	4 36	14.4	1 55	0 24	0 09	4 32	14.5	13 21

NOTE.—On this date a 125 per cent truck supply was used to eliminate delays at the mixer, because of subgrade operations. This oversupply of trucks accounts for the delays at the plant and at the site of paving. No unnecessary delays occurred at either plant or site of paving.

For two-batch heavy-duty trucks (formula 1)	For ordinary single-batch trucks (formula 2)	For two-batch high-speed trucks (formula 3)
10×2.6=26	8×2.6=20.8	6×2.6=15.6
6	4	6
Time per trip 32 minutes.	Time per trip 24.8 minutes.	Time per trip 21.6 minutes.
600÷32=18 trips. 18×2=36 batches per truck. 480÷36=13.3. As over 13 trucks are needed, 14 would be required for full delivery.	600÷24.8=24 trips. 480÷24=20 trucks.	600÷21.6=27 trips. 27×2=54 batches per truck. 480÷54=8.8 trucks. As over 8 trucks are needed, 9 would be required for full delivery.

Under field conditions heavy-duty trucks sometimes are speeded up a little and operated at a rate for which the corresponding formula is:

$$T=9d+6 \text{ - } (4)$$

This is not to be recommended, but is referred to because of the fact that it is a rather common practice and because, where it is done, it affects the number of trucks needed in order to meet specific haul requirements. With high-speed trucks the reverse is true. They are seldom, in practice, operated at a speed averaging over 17 or 18 miles an hour, which results in the approximate formula:

$$T=7d+6 \text{ - } (5)$$

Slow speed generally results from the fact that the road between the batcher plant and the mixer is commonly in such condition that higher speeds are out of the question. It is a matter of observation that contractors pay too little attention to the condition of the roads over which their trucks are operated. These are very often earth roads and heavy hauling naturally causes ruts and chuck holes. High speed, even a moderate speed, can be safely maintained only when the roads are reasonably smooth. To obtain full efficiency from the trucks available, therefore, requires that the roads be kept in the best possible condition, and this in turn requires that the provisions for road maintenance be at least as complete as any State or municipality would provide in caring for the more important roads of this type under its jurisdiction.

THE COST OF POOR ROADS

Perhaps the clearest way of presenting the financial aspects of this matter is to refer again to the fact that on a standard (5-bag) concrete paving operation the daily pay roll is likely to be in the neighborhood of $200 and the daily depreciation charge at least half as much. Time, therefore, costs the contractor about 50 cents a minute. If, then, the condition of the roads is allowed to become so bad that full efficiency can not be had from the trucks, and no extra trucks are available to make up this deficiency, there is, in addition to the direct increase in the cost of hauling, the indirect cost which results from slowing down the job as a whole—often the more important loss of the two.

Take, as an illustration, a job where the regular rate of output is 40 batches per hour. If 20 single-batch trucks are available and should, with full efficiency, deliver 40 batches an hour, but because of road conditions can only deliver 32 batches an hour, it is apparent that the cost of hauling has been increased 25 per cent. But, in addition to this, the mixer output has been reduced from 83.33 to 66.66 per cent of full production, a net loss of 16.67 per cent of the day, or 100 minutes, which, at the rate of 50 cents a minute, costs the contractor about $50 for every day during which these conditions persist.

Losses of this order are by no means uncommon though perhaps not the rule. Such as do occur, however, are commonly large enough to warrant a contractor in keeping a light grader and team at work whenever it is possible to use it to any advantage. Nor should the fact that the hauling has been sublet constitute in his mind an argument against providing this equipment and doing this work, for, as is clearly shown above, his indirect loss from slow delivery of material may easily exceed the subcontractor's direct loss from low truck efficiency and high repair costs.

The operation of a good road maintainer would hardly cost the contractor more than from $6 to $8 a day and ordinarily the whole time of this outfit would not be required in maintaining the roads. Therefore, whenever slow delivery of materials due to the condition of the roads causes loss of time at the mixer which exceeds from 12 to 16 minutes a day, giving attention to the condition of the roads is likely to prove profitable even though all hauling equipment is owned by a subcontractor. Where the delivery equipment is owned by the contractor, the reduced depreciation and repair costs which result from operation over smooth roads alone justify the cost of road maintenance and should dictate that it receive careful attention without regard to any other consideration.

SLOW DRIVING A PROMINENT CAUSE OF INEFFICIENCY

In developing the above formulas for trip time no account has been taken of the miscellaneous losses that are often noted in the operation of trucks. Table 1 is a partial study of one job which illustrates how low the efficiency of operation may drop when the supervision is weak. This loss of efficiency results from all sorts of causes, one of the most common being the interest which is taken in the repair of minor defects whenever it is learned that this excuse may make it possible for the drivers to avoid an honest day's work. Positive "hiding out" along the route is uncommon, though not unknown, but the number of little things that can be done to delay matters—a few seconds here, a few there—is surprising. None of these, however, is as effective in reducing efficiency as the driver who deliberately reduces his speed, thereby causing trucks to pile up behind him. If high efficiency is to be secured in the operation of the trucks, it is imperative that drivers be taught to drive the course in the proper time. For an occasional driver to drive faster than the established rate increases wear and tear on his truck and causes confusion without producing any corresponding advantage. To drive below the standard time causes delay.

On most jobs there are four points where the trucks stop—(1) the aggregate bins, (2) the cement sheds, (3) the turntable, and (4) the mixer. Ordinarily from a quarter of a minute to a minute is required in passing any of these four points. If then, by reason of the slow driving of one truck, the pace has been so reduced that four or five trucks arrive at the loading plant in a string, all but the first will be delayed in loading. Take a specific example, involving a 10-minute return trip, and operation at the mixer at the rate of 40 batches an hour, or one batch every 1½ minutes. Suppose the

TABLE 3.—*Stop-watch record of operation of eight two-batch trucks* [1]

Loading sand and stone		Loading cement		Turning at mixer		Dumping 2 batches		Total time required to turn, load, and dump	
Min.	*Sec.*	*Min.*	*Sec.*	*Min.*	*Sec.*	*Min.*	*Sec.*	*Min.*	*Sec.*
1	20	0	40	1	28	2	39	6	7
1	30	0	30	0	50	2	10	5	0
1	18	0	33	0	50	2	17	4	58
1	3	0	30	0	50	2	40	5	3
1	20	0	28	0	42	2	20	4	50
1	16	0	25	0	55	2	21	4	57
1	0	0	45	0	27	2	4	5	16
[2] 1	15	0	33	1	0	2	22	5	10

[1] Each entry is the average of 10 field readings. The readings show only the time consumed in the actual performance of the operations and do not include any of the various minor miscellaneous time losses indirectly associated with these operations and which can hardly be kept below a total of 1 minute per trip for this class of trucks.

[2] Average.

TABLE 4.—*Stop-watch record of operation of five two-batch trucks* [1]

Loading sand and stone		Loading cement		Turning at mixer		Dumping 2 batches		Total time required to turn, load, and dump	
Min.	*Sec.*	*Min.*	*Sec.*	*Min.*	*Sec.*	*Min.*	*Sec.*	*Min.*	*Sec.*
1	12	1	2	0	50	1	46	4	50
1	28	1	0	0	41	2	4	5	13
1	10	0	58	0	50	2	10	5	8
0	58	1	4	0	45	1	25	4	12
1	17	0	54	0	40	2	9	5	0
1	0	0	55	0	53	2	7	4	55
[2] 1	11	0	58	0	47	1	57	4	53

[1] Each entry is the average of 10 field readings. The readings show only the time consumed in the actual performance of the operations and do not include any of the various minor miscellaneous time losses indirectly associated with these operations and which can hardly be kept below a total of 1 minute per trip for this class of trucks.

[2] Average.

first truck is delayed a minute in its "get-away" from the mixer and is then driven at a rate that will require 14 minutes to make the return trip. When it reaches the cement shed it will be 5 minutes behind schedule. The next truck leaves the mixer promptly, one-half minute after the first, trailing it in. It is, then, 3½ minutes late in arriving, but, because the cement shed can load only one truck a minute, it waits there 1 minute to be loaded and leaves 4½ minutes late. The third truck, catching the first two, trails in with them, arriving 2 minutes late, and is held 2 minutes waiting to be loaded, making it 4 minutes late out of the cement shed. It will be clear without extending the illustration that, as stops occur at the batcher, at the cement shed, at the turntable, and at the mixer, the trucks must be run at a uniform rate and kept at such a distance apart that they will not lose time waiting for service at these points, if high efficiency is to be obtained.

Where trucks tend to run in bunches as above described it usually will be found that once the matter is thoroughly explained to the drivers most of them will cooperate willingly, but a few will consider the matter of no consequence and these should be summarily discharged. It is well worth while to keep a checker at the cement shed, noting time out and time back in such a way that the drivers can know whether they are driving on schedule. Really good drivers will cooperate splendidly if they have this assistance and, with other parts of the job running smoothly, can and will handle the material delivery with the accuracy of a well administered train schedule. The point is that it is no less difficult to operate material trucks to a mixer on an efficient schedule without supervising assistance than it is to operate trains without a dispatcher. The checker at the material plant should act as a dispatcher and if he has the right personality he can assist materially in keeping the trucks properly spaced and their operation generally as a basis of high efficiency.

A standard time clock can also be used to advantage on this work. Where used, it should be set ahead as many minutes as are required for the round trip. Then by stamping each driver's card as he leaves a fixed point in the circuit (the batcher plant is a satisfactory point) the driver is automatically provided with a printed record showing the exact hour and minute he should be at the batcher again. Moreover, as the card is turned in for restamping the checker can note actual time and gain or lag, so that the driver as well as the superintendent may know whether he is driving correctly or not, and if not, how much he is habitually off schedule.

The one exception to the statement that trucks should be so separated that they can be serviced as fast as they arrive at points where service must be given is found at the mixer itself. Delays at the mixer have been shown to cost the contractor about 50 cents a minute. To avoid delays at this point, therefore, the contractor may well provide something by way of insurance. If the truck supply is just sufficient to feed the mixer, the slightest delay in servicing a truck, the slightest holdup along the line, or any little thing that interferes with the continuous movement of any truck, will cause the mixer to drop a batch. To avoid this, it is desirable to operate an extra truck if two-batch trucks are in use and at least two extra trucks if single-batch trucks are employed. There should also be a stand-by truck on the job ready to run, and all drivers should have orders to abandon disabled trucks at once and return to the loading plant for a new truck, leaving the disabled truck to be taken care of by the regular job mechanics. Finally, where single-batch trucks are used, a 10 per cent surplus should generally be provided to allow for time necessarily lost in the repair shops. With these precautions, lost time chargeable to the trucks can be reduced to an absolute minimum, for it is, indeed, an unusual condition that will throw enough trucks out of use so that the delivery can not be handled by an organization of this sort.

THE QUESTION OF THE STAND-BY TRUCK

From the data given above, it is a comparatively simple matter to determine whether the truck supply available on a given job is adequate for the haul prevailing, whether the trucks in use are being operated efficiently, where losses of time are occurring, etc. But the determination of what truck supply should have been provided in the first place is a very different problem. This involves not only the haul conditions on the job but also the average haul conditions on concrete jobs in the territory where the contractor is working, the possibility of using extra trucks on other work, the cost of owning stand-by equipment, etc. A modern heavy-duty truck, for instance, will cost from $4,000 to $5,000. The yearly interest on this sum will mount to perhaps $300. Dead storage, let us assume, will cost $100 a year, and taking the truck to and from the job another $100. In the nature of the case, these figures are supposititious, for the original cost of trucks varies, interest rates vary and jobs are not all an equal distance from headquarters. However, the contractor who is endeavoring to determine what truck supply he can afford to own will know what these figures are in any individual case. For the moment it may be assumed that they generate a yearly stand-by cost of $500 for each extra heavy-duty or high-speed truck sent out to the job. To be a profitable investment, there must be reason to assume that the savings likely to develop from having such trucks will cover this charge in addition to depreciation, repairs, operating cost, and a profit while they are in use.

Now, it will be apparent at a glance that while the cost of hauling, when considered as an operation by itself and without reference to overhead, is affected by the efficiency with which it is done, it is not affected by the number of transportation units employed. In other words, if there are 1,000 hours of two-batch truck time involved in a given operation, the hauling cost per truck-hour remains constant whether the work is done in 100 days by 1 truck working 10 hours a day or whether it is done in 10 days by 10 trucks each working 10 hours a day. Moreover it may be reasonably assumed that the returns received from whatever work is done by any truck will pay for depreciation, repair costs, operating charges, etc., and will yield a profit reasonable in view of the time worked. But not all the trucks employed in a concrete paving operation are worked full time. Some are in actual use only a small part of the time, and with respect to each of these there is a short time when the truck is profitably employed and a longer period when it is idle. The question as to the advisability of adding hauling equipment which will be employed only part time resolves itself, then, into a matter of determining whether or not the increased output of the mixer made possible by having such trucks is large enough to cover the stand-by charges and show a profit to the owner sufficient to justify the capital outlay involved.

STAND-BY COST PAID BY INCREASED MIXER PRODUCTION

It has been shown that mixer time is worth about 50 cents per minute. Therefore, any saving in mixer time which will, at this rate, exceed the assumed stand-by cost of $500 a year will render the ownership of stand-by equipment profitable. For example, if having an extra truck will enable the mixer to operate more than 1,000 ($500 divided by 0.50) minutes when it would otherwise be idle, the payroll and depreciation savings thus effected will offset the truck stand-by charges. As in 1,000 minutes the full production would be 800 batches, any production in excess of this amount made possible by having the extra truck will generate a net saving, and if such a saving may be expected to accrue from year to year, as the natural result of ordinary operation, the desirability of having the extra equipment is apparent. From the theoretical standpoint, therefore, the problem is reduced to the simple one of determining whether an extra truck would increase output more than 800 batches a year. It makes no difference whether these 800 batches are hauled in one working period or in two or three. The question is solely whether the truck will enable the mixer to do the additional amount of work during an ordinary working season.

In practice, trucks sent out as stand-by equipment usually work only on the longest haul, which means that they will be called into service only for short periods when the haul is long. The analysis will, therefore, be continued on that assumption. Thus a 4-mile haul by heavy-duty, two-batch truck requires 46 minutes (see formula No. 1). At that distance, therefore, such a truck will haul 13 loads per 10-hour day; and as two batches are hauled at each load, the day's haulage will be 26 batches and 31 days' work will be required to haul 800 batches. In a similar manner it can readily be ascertained that 25 days will be required to haul 800 batches when the haul is 3 miles, and 37 days when the haul is 5 miles. If, then, the requirements are commonly such that there will be 25 days' work on a 3-mile haul, or 31 days on a 4-mile haul, or 37 days on a 5-mile haul, the stand-by cost will be covered by savings in pay roll and depreciation costs incident to more intensive operation of the mixer, and if more than this time can be worked a profit accrues. In making this analysis it is assumed, of course, that while at work the truck is making a fair profit, and that while in dead storage it is so well cared for that it does not deteriorate.

The foregoing analysis could be extended to allow for the reduction in total stand-by cost, which should properly be made when a truck is put in service. Thus, trucks used as hauling equipment on concrete highway construction ordinarily must be assumed to earn capital charges, cost of delivery to and from the job, winter storage, and profit for the whole year in addition to depreciation and operating expenses while working within the six to eight months during which they are normally operated. If, then, a truck will have three months' work, it should during that period cover at least half of the stand-by cost noted above, out of normal earnings. There is, however, no need to consider this phase of the matter further or to include it in subsequent calculations, because, as will appear later, it will be reasonably clear that trucks which are to be operated over as long a period as this should be supplied in any event, as otherwise mixer operation would fall to an obviously inefficient point. In the last analysis the real problem is that of determining when the purchase of another truck to cut down losses of time at the mixer ceases to be advisable. When that point is reached the time the truck will work becomes so short that the amount by which the stand-by cost would be offset by the earnings of the truck when actually at work is less than the probable error involved in the calculation of the stand-by cost. For that reason it should be ignored in determining whether another truck is needed.

As already noted, deductions of this sort must, of necessity, be illustrative only. Although the principles involved are constant, their application to the different conditions prevailing on various jobs produces results which vary with the local conditions. One thing, however, stands out clearly enough, namely, that if 31 days' operation on a 4-mile haul will result in savings at the mixer that will pay a year's carrying charges on a truck, there are comparatively few jobs that will not justify full hauling equipment.

The development of this phase of the transportation question from the standpoint of the single-batch truck would be based on a computation of the stand-by cost somewhat as follows:

Interest	$50
Dead storage	50
Cost of getting to and from the job	100
Annual stand-by cost	200

In this case the saving of only 400 minutes of mixer time, i. e., the delivery of 320 extra batches, will offset the stand-by cost. This is equivalent to the delivery to and the utilization by the mixer of the batches that such a truck can haul 3 miles in 15 days, or 4 miles in 19 days, or 5 miles in 23 days (see formula No. 2). For two-batch high-speed trucks the stand-by cost is about the same as for two-batch heavy-duty trucks, but the delivery per day is higher. The purchase of an additional truck is justified (by the method used above) if it can be fully utilized for 16 days on a 3-mile haul, or 20 days on a 4-mile haul, or 24 days on a 5-mile haul (see formula No. 3).

CONDITIONS AFFECTING HAUL REQUIREMENTS

With these aspects of the case in mind one naturally turns to the question—What are the ordinary haul requirements on a concrete paving job? Unfortunately sufficient data are not at hand to answer this question as fully as would be desirable. On the jobs which have been studied so far a maximum haul of 5 miles has been found to be rather common and hauls longer than this have been rare. This may, then, be taken as a reasonable basis for a further analysis of the transportation problem, and because there are very few contractors who are able to so completely synchronize the various operations involved in laying a concrete pavement as to obtain consistently an output of more than 40 batches an hour, even under a specification allowing a one-minute mix, this will be used as the basis from which to work. It will, of course, be understood that any analysis such as this must be modified by each contractor who adopts it to meet the best performance he is able to secure.

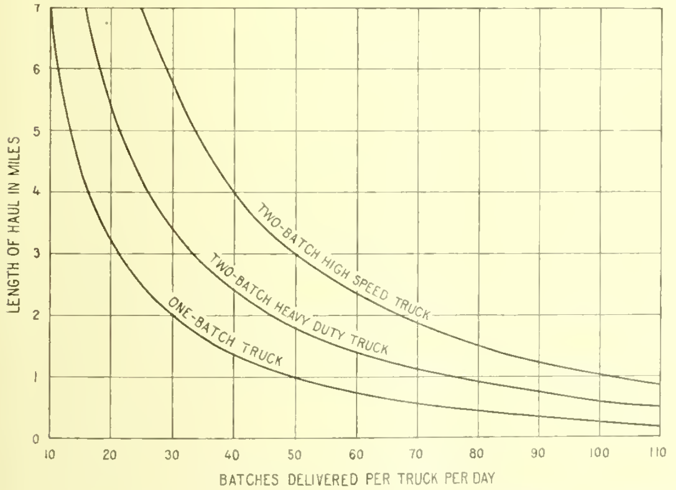

FIG. 1.—Batches delivered per 10-hour day at various haul distances by three types of truck

The mix and the cross section also are factors. Here it will be assumed that one batch (5 bags) yields 2¼ lineal feet of pavement, which means that the output per 10-hour day will be 900 feet. If this rate of production is maintained, a mile of concrete paving will be laid every 5.8 days which, for the purpose of simplifying this discussion, will be called 6 days. A full truck supply for this rate of production (40 batches per hour) for various lengths of haul is, then, as shown in Table 5, which also gives the number of days' work required of a truck in order that savings at the mixer shall offset stand-by cost, the possible saving due to additional trucks being figured at 1¼ minutes per batch at the mixer. Figures 1 and 2 develop this data in a little different fashion.

TABLE 5.—*Number of trucks required and days' work required to offset stand-by cost of various lengths of haul*

Haul	Number of trucks required			Number of days' work per truck to offset stand-by cost		
	Heavy-duty	Single-batch	High-speed	Heavy-duty	Single-batch	High-speed
Miles:						
1	6	8	4	11	6	8
2	9	13	6	18	11	12
3	12	19	8	25	15	16
4	15	24	10	31	19	20
5	19	29	12	37	23	24
6	22	35	14	44	27	28
7	25	40	16	50	32	32
8	29	45	18	57	36	36

In the analysis of any specific job, the haul and the required truck supply should be graphed in the manner used in Figures 3, 4, and 5, which were drawn from the layout data of one of the jobs studied during the past working season. These graphs are drawn to a vertical scale in which one division equals five single-batch trucks and a horizontal scale in which one division equals 1 mile of six days. If any other style of truck is to be used the vertical scale would of course be modified accordingly. In drawing these particular graphs the unit distance scale divisions have been replaced by nonuniform distance and time scale divisions, but this is a minor matter not at all affecting the actual scale used.

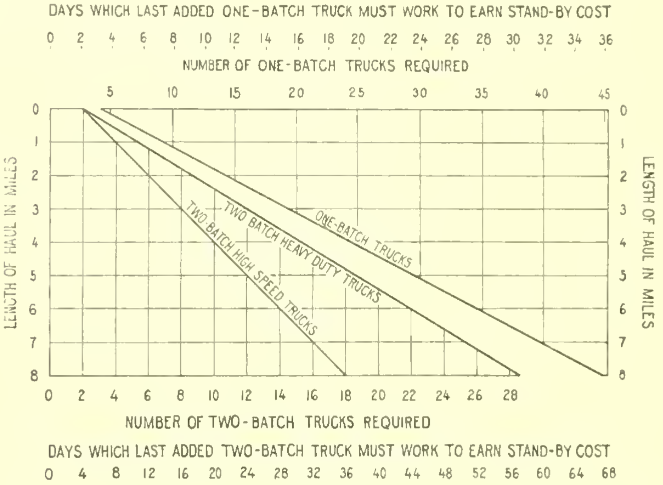

FIG. 2.—Truck requirements for various lengths of haul, and number of days trucks must work at various hauls to earn stand-by cost, with construction at the rate of one mile in six days.

To draw a chart of this sort, after selecting appropriate horizontal and vertical scales, first lay off the dead haul, that is, the distance from the plant to the road. Then from Figure 2, determine the minimum number of trucks required to handle the dead haul which determines the minimum truck supply for that set-up. The dead haul distance and the minimum truck supply having been determined and plotted (see fig. 3) the stepped line giving the points at which another truck must be added may be obtained from Figure 2 or calculated from the formula for the distance over which any given number of trucks can make a full delivery of materials.

The formulas which have been used in developing all of the graphs appearing with this article were derived as follows:

For high-speed, two-batch trucks the time required per load (formula 3) is $T = 6d + 6$. As in a 10-hour working day there are 600 minutes.

L (the number of loads hauled per truck per day) $= \dfrac{600}{6d+6}$ or $\dfrac{100}{d+1}$.

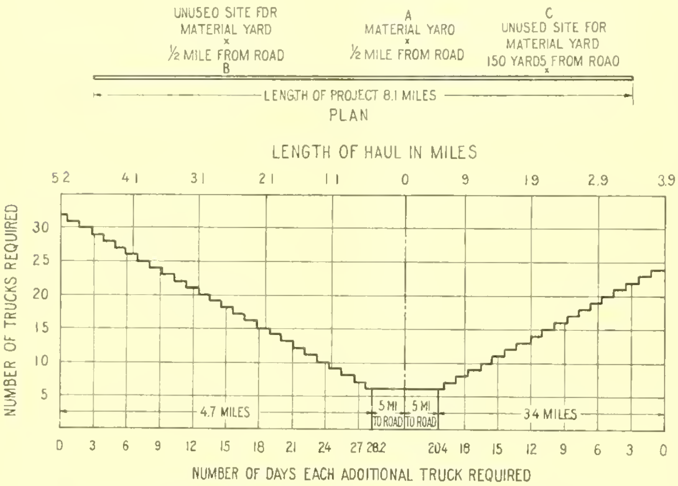

Fig. 3. Relation of location of material yard to truck supply required. This set-up was actually used by the contractor

If N equals the number of trucks, the total number of loads delivered during a working day (100 per cent efficiency in truck operation being assumed) becomes NL or $N\left(\dfrac{100}{d+1}\right)$

And as these are two-batch trucks, the number of batches deliverable per day is $2N\left(\dfrac{100}{d+1}\right)$

Now let Y equal the number of batches a day needed in order to meet regular mixer requirements.

Then, if Y equals $2N\left(\dfrac{100}{d+1}\right)$; that is, if the total delivery just equals mixer requirements, a solution of the equation for d will give the maximum distance over which full delivery can be made with any given number of trucks.

In solving this equation the value of Y is known. For 100 per cent production at the mixer and a one-minute mix, it is 480 batches. Figures 1 to 5, inclusive, were drawn under the assumption that the mixer requirement is 400 batches a day. For any other requirement the graphs would be modified accordingly. In drawing graphs such as those shown, the value of N will, of course, be varied from the number of trucks required for a zero haul, to the highest number required to obtain full production against the longest haul on the job. In plotting it should be remembered that the solution of this equation gives the longest haul against which delivery at the assumed rate can be made by the assumed truck supply, in other words, the point at which another truck should be added.

This development covers high-speed, two-batch trucks only. A similar development for heavy-duty, two-batch trucks yields the formula $Y = 2N\left(\dfrac{300}{5d+3}\right)$

For heavy-duty, five-batch trucks, the formula is $Y = 5N\left(\dfrac{300}{5d+7}\right)$ and for single-batch trucks the formula is $Y = N\left(\dfrac{150}{2d+1}\right)$

From Figure 2 it is possible to make a further analysis of the stand-by equipment problem. Thus, taking at random one example, a 16-mile job which is to be handled from two plant set-ups, from both of which the haul in each direction will be 4 miles, it is, of course, apparent from the graph for two-batch high-speed trucks that as two trucks are required at zero haul, the third must be added before full delivery of material can take place at any haul beyond the zero point. The minimum truck supply is, therefore, three trucks. These must, of course, remain in service as the haul distance increases, that is, until the maximum distance of 4 miles is reached. But 4 miles in distance has been shown above to be equal to 24 days in time. To perform the work on this 4-mile leg, these trucks must, therefore, work 24 days and as the project is, by the assumption originally made, composed of four such legs, these trucks must perform 96 days work. By a similar process of reasoning the fourth truck, required when the second half of the first mile is reached, will work 84 days, and of the two additional trucks needed to make full delivery over the next mile, one will work 72 days and one 60 days. The next two trucks will work 48 and 36 days respectively. The number of days each of these trucks will be employed is so large that, under the assumption made, there is no question that a profit will be obtained from their ownership. The real problem arises in connection with the next two trucks, those needed to secure full delivery only against the 3 to 4-mile haul. Theoretically one of these will have 24 days' work to do and the other only 12. Of the time worked by the first, 18 days will be required to offset stand-by cost, and for the same purpose 20 days' work would be required of the second. Without these trucks, the mixer efficiency would be reduced from 100 per cent at the end of the third mile to 80 per cent at the end of the fourth. If neither of these last two trucks is provided, therefore, the pay roll and depreciation losses could not exceed an average of 10 per cent, or $30 a day, over the period of 26.6 days which the work would require at the reduced efficiency—a loss of only $800.

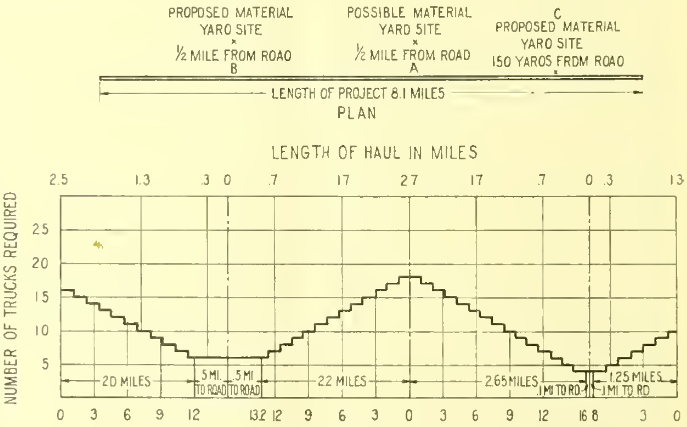

Fig. 4.—Relation of location of material yard to truck supply required (see figs. 3 and 5). If the contractor had hauled from sites B and C the cost would have been less than the cost of hauling from site A only, but more than if he had hauled from all three sites

If one truck is purchased, pay roll and depreciation losses would be reduced to an average of 5 per cent, or $15 a day, over a period of about 12.6 days—about $190. On this job, therefore, the ninth truck would produce a saving of $610, against which there must be charged stand-by cost of $500, a net saving of $110; while the purchase of the tenth truck would save $190, against which there would be the same stand-by cost of $500, a net loss of $310.

If these were the only considerations, the answer would be clear; the tenth truck would not be justified, and the ninth, while justified, would not earn enough to encourage a contractor to own it unless he has ample resources. But these are not the only matters involved. Probably a more important consideration is the fact that an efficiency of 100 per cent in the operation of trucks is almost never obtained. If, for example, only eight trucks were available for this work, there would be a period of at least 48 days during which it would be impossible to provide both a stand-by truck against emergencies and the extra hauling truck which practical experience has demonstrated is necessary if the innumerable small delays and occasional breakdowns encountered in truck operation are not to affect the output of the mixer. If the ninth truck is purchased, the period when there will be neither an extra truck in the train nor a stand-by truck will be reduced to 24 days. Just what insurance a contractor should have against truck delays will depend somewhat on what he can afford. The ninth truck will carry itself, so the insurance it gives him against day-to-day losses of one sort and another really costs him nothing. Therefore it certainly ought to be procured. The tenth does not carry itself and has reduced insurance value. To own it is not as certainly profitable. But as with other forms of insurance, its cost may serve to avoid more serious loss.

For a job of this character, it would, then seem to be desirable to provide the last two trucks, making 10 in all, and, as a matter of operating policy, to make definite plans to insure that when the long haul is reached every available truck will be in such perfect condition and the driving so carefully supervised that during the brief period when all of the transportation equipment is needed, the highest possible efficiency will be obtained. Every contractor will understand that on a job of this kind the period of long haul almost invariably follows a period of short haul during which any trucks needing the services of a mechanic can be attended to without delaying the material delivery.

HIRED TRUCKS THE BEST SOLUTION FOR EXTRA-LONG HAULS

When the problem is analyzed in this manner it becomes apparent that this general rule applies, viz, that contractors should own as many trucks as may be needed to provide full delivery up to the longest haul commonly encountered, but that the last mile of this ordinary maximum haul can reasonably be undertaken without a surplus truck in the train and without stand-by equipment in the yard.

On the other hand, there is the occasional job where the maximum haul will considerably exceed the maximum of the average job. To buy a full truck equipment for a job of this sort might, in theory at least, leave the contractor with a number of trucks on which, in ordinary years, he could not reasonably expect to earn even the stand-by cost. In a case of this kind, though special conditions may suggest a different treatment, the contractor's safest policy ordinarily lies in furnishing a standard truck equipment and renting the additional trucks needed for the extra-long hauls. While it is, of course, advisable that the hired trucks be uniform in general characteristics with those regularly used on the job, this is not necessary. Such trucks should be hired when needed and laid off as fast as the haul distance shortens. If they are hired only to meet the long-haul conditions and that fact is fully understood by the owner, the scheme can be made to work very well. Trucks for this purpose will generally cost a little more than trucks hired for the whole job, but ordinarily this is more than offset by savings in mixer time.

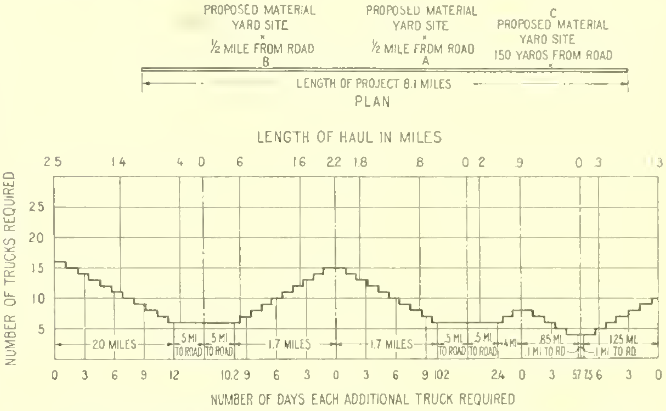

Fig. 5.—Relation of location of material yard to truck supply required (see figs. 3 and 4). If the contractor had hauled from the three sites shown, the cost of hauling would have been reduced to the lowest possible level

Take an example. The conditions on a certain job are such that the mixer must start 6 miles from the batcher plant. With a full truck supply this mixer has been producing 40 batches an hour. The contractor having found that 4 miles is his customary maximum haul has available only 15 heavy-duty two-batch trucks. But 22 such trucks are needed to deliver 40 batches an hour to the mixer against the 6-mile haul. What can he afford to pay for the 7 extra two-batch trucks or the 11 single-batch trucks that he will require to maintain his customary rate of production?

In the first place, if 22 trucks are required to produce 40 batches, 15 trucks can produce only a little over 27 batches an hour. But 40 batches per hour is an 83.3 per cent utilization of the working day, whereas 27 loads represent only a 56.2 per cent utilization. A full truck supply will, therefore, mean a saving of 27 per cent of the working day (which in this series of articles is assumed to be 10 hours long) or 162 minutes—worth, on the basis previously deduced, approximately $80. The contractor has enough trucks to maintain his customary production up to a 4-mile haul. There are, therefore, 2 miles over which losses in mixer time due to inadequate truck supply will, if no trucks are added, range from zero at 4 miles to $80 a day at 6 miles, an average of $40 per day. It has previously been shown that, at 40 batches per day, laying a mile of ordinary concrete road takes about 6 days, but laying at the rate of 27 batches will require about 9 days. The time required, without additional trucks, to lay the 2 miles furthest removed from the batcher plant will, then, be about 15 days. Having an adequate truck supply for this work should, therefore, be worth about $600. Now, as the cost of hauling is not affected by the number of units used in performing the haul, and if the contractor can employ extra trucks at a rate no higher than the cost of his own trucks, he can save all of this $600. If he has to pay a higher rate than he allows himself for his own trucks, the difference merely reduces the saving possible from operation on a basis of normal production. If the difference is greater than the possible saving—in this case $600—renting trucks for this purpose would be unprofitable.

THE KIND OF TRUCKS

Another question that presents itself is, what sort of trucks should contractors provide for this work? No effort will be made to answer this question in detail at this time but certain phases of the matter are apparent.

1. The capital outlay involved in equipping with single-batch trucks is low. On the basis of a 5-mile maximum haul and the assumptions that have been made with respect to the relative speed of the several classes of trucks, 29 single-batch trucks are needed as against 12 high-speed, two-batch trucks. The 29 single-batch trucks would cost in the neighborhood of $20,000 as against a somewhat variable maximum (depending on the make of two-batch trucks used) of perhaps $60,000.

2. The weight of a five-bag batch (sand, coarse aggregate, and cement) varies because governing specification requirements vary a good deal, but will, at times, exceed 3,500 pounds. This is too great a load for most of the single-batch trucks now in use and as a result their rate of depreciation is high. Two-batch trucks are relatively stronger and the rate of their depreciation is not visibly affected if reasonable care is exercised in their use on this work. With due allowance for an occasional factory overhauling, their depreciation should not esceed $700 to $800 a year. Single-batch trucks are commonly turned over to inferior drivers and are poorly cared for. In practice when due allowance is made for time spent in the repair shop, two and one-half single-batch trucks of the type most commonly used may be considered the equivalent of one high-speed truck. A depreciation rate of $300 per year on these trucks is, therefore, about equivalent to the rate given above for high-speed, two-batch trucks.

3. The cost of current repairs and lost time while repairs are being made appears to be a larger factor where single-batch trucks are used than where two-batch trucks are used.

4. On a job where the haul runs from practically nothing to 5 miles—to use a single illustration—the pay roll should carry an average of 16 drivers at an average rate of about $3.50 a day or $56 if single-batch trucks are used. As against this the pay roll would carry an average of eight drivers at perhaps $4 or $32 a day for high-speed, two-batch trucks. During a season of 150 working days this difference generates a pay roll saving in favor of the high-speed two-batch trucks which amounts to about $3,600. Savings in gas and oil will increase this amount to well over $5,000. This is some 12 to 15 per cent of the difference in cost ordinarily involved.

CONTRACT HAULING NOT AN UNQUALIFIED SUCCESS

Within the past few years quite a number of companies have come into existence which make hauling their business. Some of these specialize on highway construction work and within the past few years they have become a considerable factor in the concrete paving business. The theory on which they work is that as all highway work, and particularly concrete paving, is seasonal, and as the haul distances on a normal project are variable, trucks operated by a contractor must earn fixed charges during a relatively short average season. The motor truck company, on the other hand, has greater opportunity for the continuous employment of its trucks and a relatively long season over which to distribute the fixed charges.

Reverting to the case previously used as an illustration (a 16-mile project handled by two-batch high-speed trucks from two loading points), it is apparent that of the 10 trucks desirable for this work, three would be used 96 days each, one 84 days, one 72 days, and one would be used 60, 48, 36, 24, and 12 days, respectively, a total of 624 days. This amounts to an average of 62 days or, in practice, to an average of somewhat more than three calendar months of actual operation, and this is a fair year's work. To make the ownership of these trucks profitable they must, of course, cover the year's fixed charges during this relatively short working period, beside depreciation, operating expenses, repairs, etc., while working and show a profit. It is by no means true that paving contractors commonly handle more than this amount of work during a working season; that they consistently obtain other work for their trucks when they are not employed on paving work; or that the work in the hands of even the larger paving contractors is so located that an interjob transfer of even so mobile a piece of equipment as a truck, is or can be made to meet changing length of haul. The owner of a large number of trucks has, therefore, a strong argument in his favor when he offers to relieve the main contractor of the financial burden which a full equipment of trucks imposes on him. The practice of renting trucks has, therefore, grown in popularity.

Where this plan has been found in operation on going projects it has not, however, been an unqualified success. On one of the projects observed the main contractor, firmly entrenched behind a contract guaranteeing the delivery of material to him at a fixed cost per square yard of pavement laid, was thereafter indifferent as to the location of his material yard with the result that the haul was needlessly lengthened to the material detriment of the subcontractor. On another job, the main contractor installed a loading plant which operated so badly that the trucks could not make a reasonable number of trips per day and lost money from this cause. This trouble was complicated by a long delay, amounting to three weeks or more, in moving from the first set-up to a second set-up. During this period the trucks which had been working on the job were withdrawn and when the main contractor was ready to begin operations again there was a good deal of trouble in getting a new truck train. On another job the subcontractor provided trucks which were originally believed to be sufficient to meet the haul requirements but when the main contractor improved the efficiency of his operation, the subcontractor was unable or unwilling to supply additional trucks.

Other illustrations could be given but these should make the point reasonably clear, namely that, as the profit on a paving operation is so largely dependent on adequate and dependable material delivery, and the profit on the operation of the trucks is so dependent on efficiency at the loading plant and the mixer, these operations should never be allowed to come wholly into the hands of different men, for their apparent interests are bound to clash at times with the result that the profits of both suffer. This is, of course, quite a different situation than that which arises when a contractor employs a few trucks for a few days in order to maintain production against an unusually long haul. Here the amount at stake generally is not great and the employment of a few trucks to meet a special condition in no way places the contractor at the mercy of a subcontractor whose interests may at times seem to clash with his own.

SUBGRADES LIMIT USE OF HEAVY-DUTY TRUCKS

Large heavy-duty trucks offer a means of somewhat reducing the investment in trucks. A 5-ton truck will readily carry three batches and with only a moderate overload will carry four batches. A 7-ton truck will carry four batches and with moderate overload will carry five. These trucks can be reasonably depended on to average 12 miles an hour and, where loading facilities are reasonably well designed, the formulas for trip time will be as follows:

For three-batch trucks, $T=10d+9$. For four-batch trucks, $T=10d+11$. For five-batch trucks, $T=10d+14$.

If the cement is hauled in separate trucks, and a multiple-batch device is used for loading the aggregate, these formulas reduce to:

For three-batch trucks, $T=10d+6$. For four-batch trucks, $T=10d+7.5$. For five-batch trucks, $T=10d+9$.

If delivery capacity were the only consideration the large, heavy-duty truck would be widely used, as it is a dependable piece of equipment and the investment required in order to insure an adequate supply of material at the mixer against hauls of ordinary length is relatively low. It is not widely used, however, nor is it likely to be, because of its effect upon the subgrade. It takes a good while for even the best subgrade to dry out enough in the spring to carry these large trucks without serious rutting, and subgrades composed of the more plastic materials, such as heavy clays, gumbo, etc., often do not become stable enough to permit profitable operation until late in the fall. The season during which they can work without creating subgrade conditions both difficult and expensive to correct is, therefore, short. There is also to be considered the fact that whenever the subgrade is badly distorted the best attention probably does not leave it in a thoroughly satisfactory condition; and that the corrective measures tend to interfere with high production. Finally any rutting of consequence displaces so much material that the forms are thrown out of grade and alignment with the result that the contractor finds himself under a more or less constant extra expense in dealing with both the subgrade and the forms and the State or municipal organization responsible for the quality of the work, finds itself constantly in some doubt as to whether the best treatment that the contractor can give to the damaged subgrade really returns it to proper condition. The accumulation of these difficulties develops a cost, tangible in so far as it involves extra personnel, more or less intangible in so far at it adversely affects production, which measurably offsets the low delivery cost that these units show when operating over stable roads.

INDUSTRIAL RAILWAY HAULAGE

The opposite condition prevails where industrial railway is used. The general practice is to run the industrial railway on the shoulder, a practice which permits of the uninterrupted preparation of subgrade and setting of forms without danger that either will be damaged by any operation incident to the delivery of materials at the mixer. This is the greatest merit of the industrial railway system of material delivery. It is an element deserving of serious thought, for under the conditions prevailing during the spring and after any protracted rain, the subgrade is apt to be so soft that trucks of any kind distort the finished work to some extent, making it practically impossible to obtain the accuracy in the final finish of the subgrade which is theoretically desirable. Even during the drier months, from June to October, the practices which must be followed in the fine finishing of the subgrade preparatory to placing concrete, result in some irregularity and, as the operation of trucks over this finished work tends to accentuate any such irregularity, it is not practical to insist on the refinement which can be beneficially obtained when the industrial railway is used.

In spite of the advantage which the industrial railway has in this particular, its use appears to be decreasing. There are a number of reasons for this. Perhaps the most influential is the fact that, as the industrial railway is not generally used on any highway construction work except concrete paving, a contractor who invests in this form of equipment finds that a large part of his working capital is tied up in equipment serviceable only when concrete paving work can be had. Another aspect is the high cost involved. Twenty-pound track with ties, switches, etc., is worth perhaps $5,000 a mile. Engines cost from $3,000 to $6,000 each. Cars are worth about $85 each and batch boxes about $60. Necessarily these figures are general but they give some idea of the delivered cost of this equipment to the contractor. In fairly level country from 24 to 30 batches can be hauled per train and if the trains are operated at 6 miles an hour—a rather common speed—the train cycle is somewhat as follows:

	Minutes
Loading train	20 to 30
Running time (5 miles)	90 to 100
Switching at mixer	10 to 15
Unloading at mixer	35 to 45
Time per round trip	155 to 190

If the project is fairly level and production in the neighborhood of 40 batches an hour is to be obtained, it is, therefore, necessary to provide at least 4 engines, 60 cars, and 120 batch boxes, besides 5 miles of track with switches, etc. If, on the other hand, the country is hilly, the length of the trains must be cut down or helper engines put into service. Against grades of 5 or 6 per cent, which are common in highway work, a good engine will handle only 4 to 6 cars—8 to 12 batches—over track such as is commonly found on paving jobs. Where such grades will be encountered two or three extra engines must, therefore, be supplied. It is not necessary to go into the matter in greater detail to make it clear that the cost of full industrial railway equipment for a 5-mile maximum haul may, readily reach $75,000. To obtain a reasonably accurate view of the comparative investment in transportation equipment which is involved, this sum should be compared with the $20,000 required for single-batch trucks and the $60,000 required for high-speed two-batch trucks to meet the same maximum haul.

Direct delivery costs, however, are low even in rolling country. An engineer and a helper will handle a train which, in level country, may carry as much as 30 batches. Assuming that three-and-a-half trips a day are possible, this would constitute a delivery of 105 batches per crew per day. Such a crew (engine driver and helper) would cost not more than $10 per day, so it is clear that the direct labor cost involved is very low. Even when hills are encountered the labor cost would seldom be more than doubled. On short hauls the showing is not quite so favorable since the time required to load and unload the train is not affected by the length of haul. For level country, however, the average direct labor cost of delivery by industrial railway up to a 5-mile haul may drop below 8 cents a batch and even with some heavy grades may be as low as 15 cents a batch.

The actual cost of delivery, as affecting the contractor's profit and as determining what type of hauling equipment to purchase is, however, quite another matter. In its last analysis, no matter what sort of equipment is used, the cost of delivery must include all such items as: Getting equipment onto the job; special services and facilities; interest and depreciation; operating cost (labor and materials); transfer from one set-up to another on the job; and return to winter storage. These, of course, generate direct and tangible costs. But there are also certain intangible costs. If the hauling equipment breaks down there is a loss of earnings on the equipment and a cost of repairs. But there may also be a loss of output at the mixer. Commonly the crew working at the mixer and on correlated operations is of practically constant size. The pay roll is, therefore, constant. If, then, batches are dropped because the hauling equipment breaks down there is an indirect loss which may, and often does, exceed the direct losses charged against the equipment itself. No study of the relation which one style of transportation equipment bears to another is even measurably complete until it covers both of these fields, and conclusions drawn on any narrower basis are bound to lead to costly errors.

REASONS FOR DECLINING USE OF INDUSTRIAL RAILWAYS

Returning, then, to the cost of transportation by industrial railway and subjecting it to further analysis, it will be apparent that moving an industrial railway from job to job is expensive. Track, ties, switches, locomotives, cars, batch boxes, etc., to cover a 5-mile haul weigh at least 350 to 400 tons. To place any general estimate on the cost involved in moving such a weight of equipment is impossible, for the conditions governing differ too widely. However, as a common condition, getting this equipment onto the job would involve loading it onto trucks, hauling it to a railroad station, loading it onto cars, shipping it to the site of the new work, unloading it onto trucks, hauling it to the road and distributing it along the road. All of these operations cost a good deal.

Special facilities include repair shops, fuel-supply equipment, etc. Installations of this sort must be of about the same nature as on a truck job, although there is, in general, less work for the expert mechanic and more on cars, batch boxes, etc., for the blacksmith.

Interest on working capital needs no special discussion. Depreciation of equipment is rather high. Locomotives generally are of the gasoline engine type. They are well built, but suffer from wear and tear to about the same degree as heavy trucks of the same grade of construction. Track is given rough, even abusive, treatment, as are cars, batch boxes, etc. These do not wear out rapidly but careless handling and the common preference for the sky as a roof over them in all sorts of weather renders them unfit for valuable service long before they ought to go to the junk dealer. There is, therefore, no apparent reason for assuming lower rates of depreciation on industrial railway equipment than are considered applicable to other forms of heavy equipment.

The direct labor element in operating cost has been discussed. It is low. The fuel, and correlated costs, are equally low. But, in addition to the direct labor used in the operation of trains, track laying and track maintenance must be included under operating cost. The labor requirements under this heading are variable. In good weather three or four trackmen are all that are commonly used. In wet weather as many more may be needed to keep the track in passable condition. If the job is so long that the track can be used more than once, it must be taken up and relaid. The common practice is to begin operations at the point nearest the material piles, working away from them to the end of the project, laying track about as fast as the progress of the mixer requires. When the end of the first leg is reached the mixer returns to the point at which work began and works out in the opposite direction. This releases the track to be torn up and relaid as the mixer progresses.

Generally the track is torn up from the far end and hauled back by a work train. But whether this system is used or trucks or wagons are employed, it is an expensive operation. A gang of from four to six men is required to tear up and load the track and a train of cars or trucks is required to haul it, in addition to the gang used in relaying it. Finally, when the job is finished, all of the track and equipment must be accumulated and piled, and locomotives housed, pending shipment to warehouses or to the next job. All of these operations are expensive, as track is heavy and cumbersome. No very definite idea of their cost can be given in an article such as this for they are subject to so many variations from job to job that even illustrative examples are likely to prove deceptive. They have been mentioned only because no contractor can obtain even an approximate view of industrial railway hauling costs until he has given these matters consideration in the light of such experience as he may have had with this or other types of equipment.

The above deal with tangible direct costs. Besides these, the bureau's studies indicate that there are definite indirect costs to be dealt with. Thus, for instance, it is hard to get full production from the mixer when industrial railway is used. Tables 6 and 7 show readings which bring this out. The time taken in shifting trains is a common cause of mixer delays. Derailments are also a common cause; these not only cause rather frequent mixer delays, but also the spilling and loss of numerous batches. On one job which the writer visited this year a train had gone into the ditch an hour before he arrived, a second had been derailed and most of the batches spilled two or three days before, and in a third location there was evidence that another wreck had recently been cleared away. Between 30 and 40 cars were involved in these wrecks, and although exact figures as to the losses which had been involved were not available, the foreman stated that between 30 and 40 batches had been lost. These were 5-bag batches, and the cement involved alone amounted to quite a tidy sum.

TABLE 6.—*Operation time losses on industrial railway job*

Date (1925)	Proportion of working time mixer was in operation	Percentage of working time lost due to various causes: Waiting for train	Train derailed	Slow operation of mixer	Waiting for materials at railroad	Miscellaneous
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Mar. 9	28.0	45.4	26.1			1.0
Mar. 10	34.9	35.0		0.7	29.2	
Mar. 11	51.9				48.2	
Mar. 11	50.8	48.5		.7		
Mar. 12	44.6			.2	42.3	13.1
Mar. 13	54.4	28.4				18.0
Mar. 13	52.0	44.7		1.4		2.4
Mar. 14	63.4	25.6		10.7		
Mar. 16	66.1	16.7				17.3
Mar. 17	62.6	13.4				23.9
Mar. 17	44.4	54.2		1.4		
Mar. 18	30.5	69.1		.6		
Mar. 18	42.2	56.3		.5		1.3
Mar. 19	80.5	12.4				7.1
Mar. 19	58.5	40.0		.2		1.2
Average	51.0	32.6	1.7	1.1	8.0	5.7

TABLE 7.—*Operation time losses on industrial railway job*

Date, 1925	Proportion of working time mixer was in operation	Percentage of working time lost because of various causes: Handling batch boxes	Spotting cars	Switching	Mixer trouble	Slow operation of mixer	Moving mixer	Water supply	Fuel supply	Miscellaneous
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
Sept. 26	44.7	1.7	5.5		3.0	3.8	5.1		36.2	
Sept. 28	46.5	5.3		37.3			10.9			
Sept. 29	67.6	10.9		9.7			11.8			
Sept. 29	61.7	9.7		5.9		5.8	10.7			6.2
Sept. 30	43.3	6.1	1.0			4.0	20.6			25.0
Sept. 30	63.0	11.3	7.3			6.9	5.7		2.5	3.3
Oct. 1	69.2	3.2	3.1	17.9	1.9		.9		3.8	
Oct. 2	45.2	16.0	9.6		9.9	1.7			2.2	15.4
Oct. 3	41.1	1.1	18.0			4.6				35.2
Oct. 9	55.1	6.5	2.8	8.4		6.7	20.5			
Oct. 9	49.0	2.0		10.0	1.9	5.6	31.5			
Oct. 10	35.1	.9	4.8	54.1	.5	2.2	2.4			
Oct. 12	39.0	5.1		11.2		5.2	5.4			34.1
Oct. 12	55.2	10.0		22.7		7.5	2.5			2.1
Oct. 13	53.0	5.6		22.5	12.5	2.2	2.4			1.8
Oct. 14	49.4	6.6		17.9		.6	1.2	22.3		2.0
Oct. 15	60.3	11.0		13.8	5.1		9.8			
Oct. 15	40.9	1.4		10.0	1.2	20.2	26.3			
Oct. 16	65.6	3.2		7.8		2.7	20.7			
Oct. 16	40.4	4.0		2.0	39.7	1.1	2.8	10.0		
Oct. 16	77.0	2.6		.9	12.2	5.5				1.8
Average	52.5	6.0	2.5	12.0	4.2	4.1	9.1	1.5	2.1	6.0

NOTE.—No time losses because of rain or similar causes are included in this study.

HEAVY LOSSES CAUSED BY POOR TRACK LAYING

Losses of this character arise from the fact that the track is seldom given proper attention. In the first place the shoulder on which it is laid is apt to be composed of loose dirt. Such effort as is made to obtain a smooth bed on which to lay the track is likely to be confined to running a blade grader over the shoulder which, though it may leave a fairly smooth bed for the track, creates this condition by cutting off the high spots and filling the low spots. As not much is done to consolidate the loose material left in the low spots, the track initially rests on a nonuniform grade. Generally the track is then laid without much attention to exact alignment or to producing a uniform gradient. As the trains run over the track such irregularities as are permitted when it is laid are accentuated with the result that even in good weather trains have to proceed cautiously. As much of the subgrade on which the track rests is soft, it absorbs water freely during rains. Water also gets into the depressions under the ties.

It may be observed that at least some of these common conditions are avoidable. The loss of time from switching trains at the mixer is not, for instance, a necessity. It arises from the fact that switching is commonly done between the mixer and the loading bins. This, of course, requires that the empty train be pulled out before a loaded train can be run in. If the track laying is kept well in advance of the concreting and the switching is done ahead of the mixer, an adequate supply of loaded cars can be maintained at the mixer while this switching is being done. That this is not a more common practice is probably owing to the fact that the fine finishing requires more or less movement of material to and from the shoulders, and that, therefore, it is not advisable to attempt to lay track until this operation has been completed. Where switching ahead is practiced, the supply of forms must, therefore, be large and the final subgrading must be carried on much farther in advance of the mixer than is done on jobs where other forms of material delivery equipment are used.

Neither are the frequent derailments and the loss of materials necessary conditions. Track can be laid on good alignment and at a true gradient with little if any greater cost than is involved in present practices. This merely requires that the foreman in charge of track laying shall know how to lay track properly, and that he use this knowledge. The shoulder should be rolled to eliminate soft spots quite as carefully as the subgrade is commonly rolled and where the ground is likely to remain soft, ballast should be added. Finally, the track should be under constant maintenance. These operations will, it is true, increase the tangible operating cost but they will decrease the intangible cost arising from derailments and from delays in the delivery of material to the mixer, costs which are likely materially to exceed the direct cost of keeping a few men at work on track maintenance.

Delivery by wagon to the job, where materials were piled along the subgrade and later moved to the mixer in wheelbarrows, was at one time the standard practice. It has now been all but abandoned. The fact that more or less material was lost was a factor, but the great objection to this method lay in the poor proportioning which resulted from using wheelbarrows as measuring boxes.

LOCATION OF LOADING PLANT IMPORTANT

From the above discussion it will be apparent that in any effort to develop a high rate of production the handling of materials is an important consideration. It must also be apparent that the cost of hauling is a large element in the total cost of paving work. Indeed, it is so high that the location of material piles becomes a matter of primary consideration if profitable operation is to be insured. Just how essential the proper location of the material loading plant is has been shown graphically in Figures 3, 4, and 5. It may be further developed by a mathematical study of the effect of distance on the hauling cost. Thus, by reference to Table 5 and to formula No. 2, it will be seen that on a basis of full operating efficiency something over 30 single-batch truck-days are required for the delivery of the materials for an average mile of ordinary 18-foot pavement. If to this is added the extra truck or two needed to insure against delays, the truck time involved will be seen to average about 35 days per mile of pavement per mile hauled. A fair allowance for single-batch trucks (driver, oil, gas, and tires, repairs, depreciation, profit, etc.) is from $9 to $10 a day, which results in a hauling cost of from $315 to $350 per mile. Necessarily these figures are approximate, for various factors will affect both the number of truck days and the actual average cost of operating a truck, but they are reasonably accurate for the conditions assumed and sufficiently so to illustrate the point involved, namely, that cost mounts rapidly with the distance materials must be moved. With this as a basis, it will be obvious that haul develops cost about as follows:

Miles of pavement	Cost of hauling for each mile	Total cost of hauling to end of each mile	Miles of pavement	Cost of hauling for each mile	Total cost of hauling to end of each mile
First	$350	$350	Fifth	$1, 750	$5, 250
Second	700	1, 050	Sixth	2, 100	7, 350
Third	1, 050	2, 100	Seventh	2, 450	9, 800
Fourth	1, 400	3, 500	Eighth	2, 800	12, 600

The above development is for single-batch trucks. A table for any other style of truck would show the same characteristics. With industrial railway, because train operation is so small a factor in total operating cost, a development of this sort does not apply. Cost, with this form of transportation, is more nearly uniform.

Where a job layout is under consideration the cost of haul must be equated against the cost of moving the material handling plant. Take a typical case as shown in Figures 3, 4, and 5, which is from a project that roughly parallels a railroad and is 8.1 miles long. Station A is 3.4 miles from the south end of the road and half a mile off the road. Station B is 6.1 miles from the south end of the road and also one-half mile off the road. Station C is 1¼ miles from the south end of the road and only 150 yards from the road. All existing roads are unimproved. Should the contractor (1) make one set-up at A, (2) a set-up at B and one at C, or (3) a set-up at A, one at B, and one at C. The set-up at A would involve a 5.2-mile haul north and a 3.9-mile haul south from the set-up which by reference to the above figures will be seen to involve hauling costs as follows:

South from A, 3.9 miles	$3, 360
North from A, 5.2 miles	5, 670
Total	9, 030

The layout involving set-ups at B and C will involve hauling costs as follows:

South from B, 2.7 miles	$1, 785
North from B, 2.5 miles	1, 575
South from C, 1.3 miles	560
North from C, 2.7 miles	1, 785
Total	5, 705

This arrangement of the haul is preferable to the first if the move from B to C can be made for less than $4,325. The layout involving set-ups at A, B, and C will involve hauling as follows:

South from B, 2.2 miles	$1, 260
North from B, 2.5 miles	1, 575
South from A, 0.9 miles	315
North from A, 2.2 miles	1, 260
South from C, 1.3 miles	560
North from C, 0.9 miles	315
Total	5, 285

The third arrangement is preferable to the second if the third set-up can be made for less than $420 which is not likely. Contractors face such problems as this on almost every job undertaken and more often than might be supposed a failure to properly analyze the situation results in dropping profits that, with a little careful thought, could easily have been saved.

Besides the difference in haul and hauling cost that different arrangements of the loading points yield, these different plans present very different maximum hauling equipment requirements. This also is an important matter. To insure full delivery of material to the mixer under the first plan more than 30 trucks would be needed. Under the second plan the maximum haul is only 2.7 miles and 18 trucks would meet all requirements. Moreover under this plan the average utilization of the trucks would be relatively high because there are four points of maximum haul—three of about the same length, as compared with two in the first case. It is not necessary to extend this discussion to justify the statement that the cost of haul increases so rapidly as the length of haul increases, and the number of trucks required for high production is so affected by long haul, that careful consideration should be given to this point, not only by contractors but also by those responsible for planning and locating highway work of this character.

(Continued from page 219)

In West Virginia and Kentucky corporate ownership has apparently not developed to the stage reached by Maryland and Connecticut. In each of these States the individuals and partnerships operate 64 per cent of the busses. This condition is reversed again, however, in Washington, which is more similar to the first two States, the corporations, which are only 30 per cent of the number of operators, controlling 55 per cent of the busses. The individual operators in this State own an average of 2.2 motor busses, the partnerships 3.1, and the corporations an average of 6.9.

The tendency toward consolidation has proceeded further in Maryland than in the other States, and the data available for the two years, 1924 and 1925, indicate the rapidity of the development. The 106 busses in 1924 were operated by 43 owners; by 1925 the number of busses had increased to 161 with the addition of only one to the number of operators. Eight operators owned approximately 50 per cent of the busses in 1924, and by 1925 the same percentage was in the hands of only three operators. However, there were just as many small operators in the latter year, and the increase in the number of busses reflected the greater number of busses owned by a few operators. The change came about largely as the result of the expansion of the business of one company, which in 1924 owned only 13 busses and by 1925 had increased the number of its vehicles to 57. This company has operated this year over 15 distinct and separate routes, varying in length from 12 to 71 miles.

The studies that have been made indicate clearly that the motor vehicle is rapidly becoming an important agency in the common-carrier service of the public, involving, of course, an increase in the importance of the highways which serve as the avenues for this new type of public service. The studies inspire a feeling of confidence that the motor bus, as well as the motor truck, is capable of becoming an invaluable ally of the rail carriers by making it possible to reach sections of the country not supplied with railroad facilities and by facilitating the movement of passengers over short distances where rail service is to a certain extent slow and inconvenient.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets or to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
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*370. The Results of Physical Tests of Road-Building Rock. 15c.
386. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage and Revenues in the United States, 1914. A Summary.
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670. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
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*704. Typical Specifications for Nonbituminous Road Materials. 5c.
*724. Drainage Methods and Foundations for County Roads. 20c.
*1077. Portland Cement Concrete Roads. 15c.
*1132. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c
1216. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
No. 1259. Standard Specifications for Steel Highway Bridges adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULAR

No. 94. TNT as a Blasting Explosive.
331. Standard Specifications for Corrugated Metal Pipe Culverts.

FARMERS' BULLETINS

No. *338. Macadam Roads. 5c.
*505. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *727. Design of Public Roads. 5c.
*739. Federal Aid to Highways, 1917. 5c.
*849. Roads. 5c.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D-4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

NOVEMBER 30,1925

STATES	FISCAL YEARS 1917-1925 PROJECTS COMPLETED PRIOR TO JULY 1, 1925			FISCAL YEAR 1926 PROJECTS COMPLETED SINCE JUNE 30, 1925			FISCAL YEAR 1926 * PROJECTS UNDER CONSTRUCTION			FISCAL YEAR 1926 PROJECTS APPROVED FOR CONSTRUCTION			BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863,197.86	611.8	$ 6,001,084.21	$ 2,842,164.67	341.4	$ 9,558,742.98	$ 4,664,244.44	488.0	$ 953,521.33	$ 431,936.34	28.2	$ 2,007,112.69	Alabama
Arizona	9,580,133.43	5,016,119.94	613.8	340,190.05	207,890.13	21.7	1,940,864.50	1,241,011.21	151.3	176,977.10	108,150.70	26.7	1,988,169.02	Arizona
Arkansas	13,310,190.08	5,380,181.73	1046.9	2,782,260.41	1,180,673.76	101.4	6,608,260.60	3,173,371.20	452.8	490,437.49	212,838.99	31.2	390,831.32	Arkansas
California	22,346,175.99	10,719,249.61	894.8	2,036,363.75	911,402.63	47.4	11,468,928.57	5,666,810.36	354.2	108,448.06	51,570.18	1.8	2,239,076.22	California
Colorado	11,876,703.94	6,067,814.34	651.2	857,420.69	459,584.79	35.8	4,513,742.94	2,258,829.88	239.9	109,339.83	61,331.34	7.1	2,097,867.66	Colorado
Connecticut	4,558,639.29	1,819,368.66	101.6	750,966.91	250,212.14	13.4	1,901,816.16	590,259.68	29.2	15,816.91	7,907.95	0.1	1,192,504.57	Connecticut
Delaware	4,281,559.81	1,496,190.66	107.1	455,744.13	212,544.95	12.3	980,042.66	388,784.75	22.7				10,912.65	Delaware
Florida	2,969,273.72	1,405,487.97	96.3				8,970,318.13	4,370,709.60	251.8	1,991,337.04	650,530.77	41.6	761,040.66	Florida
Georgia	20,156,002.37	9,406,366.46	1478.3	2,241,919.32	1,084,752.54	131.6	11,441,723.16	5,360,408.07	725.2	688,487.19	274,351.85	59.9	324,885.08	Georgia
Idaho	9,394,676.80	4,815,332.26	600.1	211,619.32	121,958.48	24.6	3,184,302.51	1,997,516.39	206.0	603,213.91	359,940.73	16.1	328,290.14	Idaho
Illinois	40,010,481.10	18,640,076.28	1236.2	688,967.89	340,741.22	25.0	8,644,746.27	4,189,261.40	290.9				3,486,503.10	Illinois
Indiana	13,638,172.66	6,562,455.68	422.1	1,132,976.57	544,542.84	41.2	17,995,239.73	8,445,342.09	492.9	543,575.62	269,346.52	16.6	446,777.87	Indiana
Iowa	27,272,285.21	11,107,492.99	1996.9	722,462.52	320,365.06	36.6	7,747,366.71	3,485,669.59	476.7	1,074,988.86	514,403.90	125.2	1,997,162.46	Iowa
Kansas	26,399,695.77	9,755,273.32	831.4	3,168,308.25	1,386,657.00	178.3	10,429,367.14	4,429,015.72	499.7	3,080,125.20	690,874.84	171.8	1,130,424.12	Kansas
Kentucky	14,832,324.28	6,205,994.59	584.9	1,556,093.80	663,291.14	46.9	8,207,285.81	3,679,390.14	323.8	352,618.16	176,309.07	16.8	1,071,015.06	Kentucky
Louisiana	11,939,424.97	6,279,870.86	927.6	876,723.58	411,312.04	64.9	3,115,016.87	1,495,944.69	139.6	457,859.71	228,929.84	7.2	855,686.57	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	249,291.85	123,602.06	9.8	2,046,840.72	825,486.82	66.2	188,780.86	94,390.42	8.3	829,904.38	Maine
Maryland	8,132,506.90	3,849,383.16	294.4	368,826.12	177,191.16	14.8	2,814,750.06	1,244,591.15	126.2				19,267.55	Maryland
Massachusetts	14,047,656.22	6,467,661.28	300.6	751,829.46	271,111.11	20.4	6,104,839.26	1,591,752.67	80.4	323,228.34	105,000.00	5.2	1,584,145.94	Massachusetts
Michigan	16,234,000.80	7,328,316.91	612.6	5,743,878.35	2,637,450.74	226.2	11,779,311.22	5,400,339.73	316.8	309,916.00	113,210.57		2,645,629.06	Michigan
Minnesota	30,416,685.89	12,738,642.04	2721.2	3,969,203.46	1,706,143.23	339.2	7,734,594.96	2,923,900.00	564.8	522,061.23	26,900.00	30.4	66,026.73	Minnesota
Mississippi	10,292,286.79	4,988,702.73	803.4	1,442,014.26	720,045.42	110.0	8,430,582.28	4,209,035.47	448.4	754,624.88	377,312.42	38.1	539,718.96	Mississippi
Missouri	17,368,156.67	8,219,411.43	1118.9	3,105,119.51	1,493,061.27	132.7	26,337,684.17	10,386,545.98	755.7	1,134,406.92	48,617.89	29.2	231,952.43	Missouri
Montana	10,156,600.41	5,317,523.16	921.6	607,162.46	474,072.98	57.9	1,848,094.86	1,344,221.16	186.3	1,003,571.06	544,458.41	117.9	4,192,949.30	Montana
Nebraska	9,306,374.36	4,389,523.50	1570.6	1,440,738.00	696,928.38	117.0	10,537,597.85	5,215,015.87	1066.0	818,557.69	407,874.58	59.8	2,337,754.67	Nebraska
Nevada	4,917,465.69	3,088,299.78	357.3	988,525.19	828,609.21	79.7	4,563,690.51	3,816,052.30	412.4				113,935.71	Nevada
New Hampshire	4,165,687.86	1,986,226.87	208.1	331,478.58	154,302.87	11.9	1,060,881.86	493,891.94	33.5	38,465.92	19,232.96	1.6	150,212.36	New Hampshire
New Jersey	11,961,357.46	3,820,679.99	219.1	1,256,602.18	295,303.47	16.0	9,432,218.78	3,366,755.71	75.5				50,972.83	New Jersey
New Mexico	8,717,999.18	4,914,070.61	1081.3	2,836,457.02	1,885,680.73	253.4	2,054,029.17	1,300,477.77	191.4	167,064.76	129,453.38	6.6	1,555,539.51	New Mexico
New York	28,597,769.67	12,229,076.53	831.5	4,730,997.05	1,888,710.69	118.2	33,931,726.76	10,645,426.65	667.3	9,674,570.00	2,282,645.00	152.5	3,352,170.23	New York
North Carolina	21,014,450.41	8,746,464.69	1119.8	2,583,206.80	1,061,531.74	67.1	8,483,619.06	3,596,828.11	202.5	1,229,083.60	570,253.57	34.4	33,583.99	North Carolina
North Dakota	10,829,263.82	5,268,930.47	1917.6	604,824.27	302,281.60	142.2	4,449,468.93	2,226,648.90	554.9	722,658.57	361,329.20	73.5	1,395,748.83	North Dakota
Ohio	41,572,252.81	15,244,993.93	1191.1	2,193,511.12	796,893.17	66.9	11,985,355.41	4,525,272.20	363.3	1,880,672.34	631,260.22	50.5	1,756,339.48	Ohio
Oklahoma	20,787,024.94	9,672,890.34	852.2	1,933,800.16	883,457.45	87.2	6,950,244.64	3,316,166.99	316.2	532,795.73	136,009.35	17.1	299,017.87	Oklahoma
Oregon	14,388,168.70	7,142,364.63	794.6	1,122,747.89	605,402.45	59.2	3,085,405.48	1,806,636.18	160.3	287,542.20	5,000.00	8.8	136,998.74	Oregon
Pennsylvania	43,054,835.19	16,222,023.97	850.3	3,453,565.26	1,039,506.00	62.8	36,843,299.82	9,975,777.80	679.8	1,347,522.21	383,823.01	25.7	370,730.22	Pennsylvania
Rhode Island	2,628,496.20	1,119,688.09	64.8	481,300.30	154,988.63	6.0	2,017,073.06	519,408.63	31.2	625,956.10	197,730.00	13.2	310,128.65	Rhode Island
South Carolina	11,163,347.84	6,121,267.54	1235.9	1,051,972.51	357,678.74	58.6	6,130,486.86	2,885,364.82	345.4	1,694,666.40	380,456.51	73.2	4,763.39	South Carolina
South Dakota	12,091,434.67	5,989,879.00	1447.9	1,740,221.48	845,568.60	221.6	6,468,920.79	3,049,699.59	920.8	22,452.56	11,226.27		48,218.64	South Dakota
Tennessee	13,789,140.98	6,732,079.77	437.9	2,555,128.53	1,261,638.03	84.4	10,426,657.32	4,649,550.28	348.4	770,076.60	327,879.90	36.7	691,024.02	Tennessee
Texas	54,120,970.83	21,057,940.12	3907.1	7,549,615.16	3,051,432.81	461.1	21,225,580.96	9,373,403.32	1212.0	4,631,471.47	2,111,613.38	243.3	579,124.37	Texas
Utah	6,259,153.41	3,818,836.91	423.1	132,178.99	99,880.35	5.9	3,246,305.87	2,209,280.96	256.6	283,128.90	215,005.61	15.9	627,524.17	Utah
Vermont	3,015,174.61	1,462,894.46	107.8	187,505.36	93,686.98	3.3	1,976,543.54	876,638.12	46.4	59,192.10	29,596.05	0.1	450,066.40	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	5,632,903.03	2,640,266.28	207.2	6,215,080.97	2,732,240.28	214.1	732,786.55	336,544.40	23.2	74,612.84	Virginia
Washington	13,352,504.18	6,117,211.87	526.7	2,429,355.02	1,048,887.39	98.6	2,948,073.63	1,192,400.00	65.1	731,401.44	250,000.00	0.5	407,196.74	Washington
West Virginia	7,343,200.86	3,230,293.33	326.7	252,664.30	123,013.89	12.8	6,979,050.76	2,787,834.00	185.6	777,918.20	298,915.54	28.7	118,518.24	West Virginia
Wisconsin	21,807,140.91	8,919,640.62	1451.7	374,997.50	182,317.55	29.3	6,167,346.48	3,035,258.40	297.5	83,474.08	41,735.00	6.5	3,389,601.43	Wisconsin
Wyoming	8,809,819.33	4,739,096.67	982.0	854,988.57	521,673.58	80.3	3,539,231.43	2,224,841.24	231.3	6,265.60	4,022.00		141,046.51	Wyoming
Hawaii							342,277.22	37,440.00	6.5				637,088.00	Hawaii
TOTALS	740,140,790.82	325,654,346.00	41838.3	86,838,710.11	39,360,311.93	4383.7	394,870,629.38	169,285,752.15	16576.4	41,901,058.30	14,473,918.66	1651.1	49,469,671.36	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 115,379,382.03 Federal aid $ 50,873,266.64 Miles 4640.1

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 6, NO. 11 ▽ JANUARY, 1926

SIXTY PER CENT OF ALL MOTOR VEHICLES ARE OWNED IN CITIES

WASHINGTON : GOVERNMENT PRINTING OFFICE 1926

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

U. S. DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

VOL. 6, NO. 11 JANUARY, 1926 H. S. FAIRBANK, Editor

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URBAN ASPECTS OF HIGHWAY FINANCE[1]

By JACOB VINER, Professor of Political Economy, University of Chicago

This study of the problems connected with the financing of highways which affect most closely and peculiarly the governments and residents of urban communities was made especially for and under the auspices of the Highway Research Board of the National Research Council. In 1924 a committee of the National Tax Association, of which Professor Viner was a member, presented a comprehensive report on the problems of financing rural highways, but in accordance with its instructions it excluded consideration of the specially urban aspects of the problem from its study.

The present study is, therefore, in a sense a supplement to the former one and deals only with problems not considered previously. The report of the study is divided into three parts, of which the first, dealing with the special interests of cities and urban motor-vehicle owners in the highway finance policies and practices of State and county governments, is published in this issue. The second, dealing with the methods and problems of urban highway finance, and third, dealing with certain financial aspects of the traffic-congestion problem, will be presented in a future issue.

THE tremendous growth in recent years of motor transportation has made necessary a great increase in expenditures for rural highways. Such expenditures for the United States as a whole now approximate $1,000,000,000 annually, and appear to be stabilized for the time being at this level. To a large and growing extent the funds for these expenditures are obtained by the special taxation of motor vehicles, in the form of Federal excise taxes on motor vehicles and parts, passed on in part in the form of Federal aid to the States for highway purposes, and of State motor-vehicle license and fuel taxes. In 1924 the special taxation of motor vehicles produced revenues equal to about 45 per cent of the total expenditures of the country as a whole for the construction and maintenance of rural highways, exclusive of interest on highway indebtedness,[2] and the percentage has been steadily rising each year. It will in all probability reach 50 per cent in 1925.

The Federal excise taxes and the State motor-vehicle license and fuel taxes within each State apply equally to automobiles of the same class regardless of whether they are owned and used in cities or in rural areas. The other sources of rural highway revenues are predominantly State and county ad valorem taxes on general property and bond issues, and the bond issues will in the main be redeemed with funds derived from property taxation or from special taxes on motor vehicles. State and county levies on general property are, with only two exceptions of importance,[3] applied to urban and rural population at uniform rates within each taxing district, and in these two instances the rates are higher on urban than on rural property.[4] It follows that urban dwellers and urban vehicle owners make the same contribution per unit of property and per vehicle of the same class to the cost of financing rural highways as do rural dwellers. It is possible from available data to estimate roughly the relative amounts contributed to the costs of rural highways in the form of motor-vehicle taxes by rural and urban vehicle owners.

URBAN-OWNED MOTOR VEHICLES 60 PER CENT OF TOTAL

The Federal census for 1920 shows that on January 1, 1920, there were 2,285,531 automobiles and trucks on farms.[5] The total registration of motor vehicles for the country as a whole in the course of 1919 was 7,565,446, and in the course of 1918 was 6,146,617.[6] Assume that one-seventh of the vehicles registered in 1918, or 878,088, were scrapped during 1919. This would make the number of vehicles in existence on January 1, 1920, 6,687,358, or the registrations during 1919 minus the number scrapped during 1919. On this basis the proportion of urban to total vehicles on January 1, 1920, was approximately 66 per cent. Some allowance should be made, however, for automobiles owned by rural dwellers who are not farmers and by residents in villages and small towns properly to be included as part of the rural area. It is estimated that this would reduce the percentage of urban-owned to total vehicles to 60 per cent, and it would be inferable that urban vehicle owners contributed in about the same proportion to the motor vehicle tax revenues.

To obtain an exact figure, further corrections would have to be made. The average motor-vehicle tax paid per vehicle is much greater for trucks than for passenger cars,[7] and on January 1, 1920, only 6 per cent of the motor vehicles on farms were trucks,[8] as compared to 11.7 per cent of the registrations in 1919 in the United States as a whole.[9] Variations in tax rates combined with variations in percentages of urban to total vehicles as between different States, possible variations in the average payments of gasoline taxes between rural and urban vehicles owing to different annual mileage per vehicle of these two classes of vehicles, and possible variations in the average license fees paid by rural and urban passenger cars because of differences in the type of vehicle commonly owned by rural and urban dwellers, respectively, are further factors affecting the estimate here made of the proportion of the motor vehicle tax revenues paid by urban-owned vehicles. These factors, however, probably tend in the aggregate to increase rather than decrease the proportion contributed by urban vehicle owners. If there has been no substantial change since 1920 in the proportions of rural to urban vehicles, the estimate of 60 per cent as the proportion of motor vehicle tax revenues contributed by urban vehicle owners is a conservative minimum estimate. On this basis of calculation, urban motor-vehicle owners contributed approximately

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[1] Part of a report presented by the writer before the annual meeting of the Highway Research Board, National Research Council, at Washington, D. C., December 3, 1925.

[2] National Automobile Chamber of Commerce, Facts and Figures of the Automobile Industry, 1925 edition, p. 49.

[3] Minnesota and North Dakota.

[4] There are often substantial differences, however, between the ratios of assessed to true value for urban and rural property, respectively, with the result that the effective rates are different though the nominal rates are equal. Whether for the country at large such differentiation in assessment ratios operates in favor of or against urban property it is impossible to decide from the scanty evidence available.

[5] Fifteenth Census of the United States, Vol. V, p. 514.

[6] National Automobile Chamber of Commerce, Facts and Figures of the Automobile Industry, 1925 edition, p. 5.

[7] A comparison for 1924 for 28 States, made by Dr. Henry R. Trumbower, showed an average license fee of $10.70 per passenger car as compared to $21.90 per truck. (Proceedings of the Fourth Annual Meeting of the Highway Research Board, 1925, p. 86.) As trucks ordinarily consume more gasoline per mile than do passenger cars, the contribution to gasoline taxes is also probably greater per car for trucks than for passenger cars.

[8] Fifteenth Census of the United States, Vol. V, p. 514.

[9] Facts and Figures of the Automobile Industry, 1925 edition, p. 4.

$260,000,000 in motor-vehicle taxes to the financing of rural highways in 1924.

URBAN USE OF RURAL HIGHWAYS

It is protested in many quarters, and especially by municipal officials, that this is an inequitable situation and that urban dwellers should not be required to pay the bulk of the costs of rural highways, especially since rural dwellers make little or no direct contribution to the costs of urban streets. The volume of protest is steadily growing, and, as will be shown later, it has already resulted in some States in a measure of reorganization of the highway finance relations of State and city, and county and city governments. Defense of the maintenance of the existing relationship is most conspicuous on the part of the motor interests. They oppose any diversion of highway revenues from the State or county treasuries to the municipalities on the grounds that: (1) It would tend to retard the State and county programs of highway construction and maintenance; (2) the highway improvement program of cities is less elastic and flexible than the rural highway program, and the cities will procure funds by some means or other to carry out at least the major elements of their programs; and (3) the transfer of highway revenues from the State and county treasuries to the municipalities will not relieve the urban motor-vehicle owners as such of any special tax burdens which they already bear, since urban streets are now financed only to a negligible degree by special municipal taxation of motor vehicles. These arguments, and especially the first two, have a considerable measure of validity, but they appeal to reasons of expediency and not to the fundamental equities in the situation.

To the protest that it is inequitable that urban property owners and urban motor-vehicle owners should be forced to contribute to the cost of rural highways at the same rates per unit of property or per vehicle as rural property or vehicles, while, on the other hand, rural property and rurally owned motor vehicles are not required to make any direct contribution to the costs of city, a more cogent reply could be made if it could be demonstrated that traffic on rural highways consists predominantly of urban vehicles, whereas but a slight percentage of the traffic on city streets consists of rural vehicles. Statistical data on these points appear to be sadly lacking. Of the many city traffic surveys which were examined, there was not one which attempted an estimate of the percentage of motor traffic on city streets which consisted of foreign vehicles. Municipal highway officials who were consulted confirmed, however, the common impression that the great bulk of the traffic on the streets of the large cities consists of local vehicles, and that the percentage of rural vehicles on the streets of great cities is at any one moment negligible.

For the percentage of urban vehicles on rural highways the lack of statistical data is almost as complete. A test count made in 1922 on the rural highways of Davidson County, Tenn., in which county Nashville is situated, showed that 70.4 per cent of the vehicles were city owned as compared to 29.6 per cent owned in the country. An inquiry made under the same auspices indicated that the annual mileage per vehicle on rural highways was for urban-owned vehicles 40 per cent that of rural-owned vehicles.[10] The percentage of urban vehicles to total vehicles on the rural highways should be higher in the immediate neighborhood of cities than on portions of the rural highway system distant from any city. On the other hand, the annual mileage of urban vehicles on rural highways should be greater for vehicles owned in the smaller cities than for vehicles owned in the great cities, where the mileage of street pavements is greater and access to the rural highways is ordinarily more difficult.

The Davidson County, Tenn., data can not be applied, therefore, to the metropolitan problem without important qualification, but they do indicate that a substantial fraction of the traffic on rural highways consists of urban-owned vehicles.[11] Confirmation is supplied by the results of an earlier test made on the rural highways of Iowa, which has no large cities. On these rural highways test counts indicated that intracounty traffic from town to town plus circle traffic from town into country and back to town was 30.4 per cent of the total traffic. To obtain the total percentage of the traffic which consisted of urban cars it would be necessary to add (1) urban vehicles going from town to farm, urban vehicles going from farm to town, (2) and urban vehicles on longer trips crossing county and State boundaries, for which data were not separately gathered.[12] Search for further data on this point was unproductive of results.

TRAFFIC COUNTS SUGGESTED AS BASIS OF COST APPORTIONMENT TO RURAL AND URBAN USES

If the principle be accepted that the costs of financing rural highways should be borne by the users thereof and should be apportioned among the different classes of users in proportion to their use thereof, urban and rural motor vehicles should contribute to the costs of rural highways in the proportions of their respective average annual mileages on such highways. It would probably be impossible in practice to secure comprehensive and unbiased mileage data of this character if it were to be used as a basis for the apportionment of motor-vehicle taxes, but the same purpose could be adequately served if at periodic intervals traffic counts were made on rural highways to ascertain the relative proportions of urban and rural traffic and the costs of rural highways were met from motor-vehicle taxation apportioned to urban and rural areas accordingly.[13] If it were disclosed by such traffic counts that the relative use of rural highways by rural and urban vehicles was substantially different from their relative contributions to motor vehicle tax revenues, adjustment should not be made by differentiating in the rates of State taxation as between urban and rural vehicles, since such differentiation would open the path to serious political dangers, and in any case would probably be held unconstitutional in most of the States.

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[10] University of Tennessee Engineering Experiment Station, "Highway Economics and Highway Transport in Typical Counties of Tennessee," 1922, p. 19.

[11] The study of traffic on the highways of Cook County, Ill., recently made by the Bureau of Public Roads and the Cook County Highway Department shows that the great bulk of traffic on the county highways is produced by Chicago and towns located within 5 miles of the city limits. The report of this study will be published shortly by the Cook County Highway Department.—*Ed.*

[12] T. R. Agg, "Traffic on Iowa Highways," Bulletin 56, Iowa State College of Agriculture and Mechanical Arts, Engineering Experiment Station, Jan. 21, 1920.

[13] Though contrary opinions have been expressed, a count of the relative numbers of vehicles of different types which pass the counting stations during the test period will, if the stations are sufficiently numerous and are satisfactorily distributed, account for rleative mileage of the different types of vehicles on the highways in question. The greater the mileage during the test period of any vehicle, the greater the likelihood that it will pass a given counting station. If the information needed, as in this case, is relative mileage, counting the relative numbers of the vehicles passing the counting stations will provide it. If what is wanted is the relative numbers of vehicles of different types on the highways at a given moment, the proper test is an actual count of the numbers of each type to be found on selected stretches of highway at that moment. In each case, of course, the results obtained are merely an index of the situation whose accuracy is dependent on the accuracy of the count, the sufficiency in number and in location of the counting stations, the degree to which conditions at the test period are representative of conditions at other times, and other such factors.

A more desirable procedure would be, in case it were found that the contributions of urban vehicle owners were more than proportionate to their use of rural highways, to refund to the city treasuries from the motor vehicle tax revenues sufficient to equalize the ratios of contribution to the ratios of use. On the other hand, if it should be disclosed that urban vehicles are contributing less than in proportion to their use of the rural highways, which in the light of the scanty evidence available scarcely seems likely, adjustment could be made by contributing more generously out of the State revenues to the cost of rural local or secondary highways used mainly for local rural traffic. In all cases account should be taken of the relative contributions of urban and rural areas to the State and county highway funds by means of property and other taxes as well as by means of motor-vehicle license and gasoline taxes. As a rule, road district taxes and special assessments are in rural areas now used only in the financing of local roads which serve primarily local needs and are not used to any appreciable extent by urban vehicles. But where primary highways are financed by these methods credit should be given to the rural areas for their contributions in this form.

STATE AND COUNTY GRANTS TO CITIES FOR HIGHWAY PURPOSES

In a number of States grants are made by the State out of its highway funds to the cities for use in financing city streets. The treatment in this respect of large cities is often different from that accorded to small towns, and we will deal first with the cases in which cities over 30,000 in population receive State aid for street purposes from the State, either immediately or intermediately through the counties, and either in the form of appropriated grants or by permitting the cities to share in the motor vehicle tax revenues. Most of these instances are of very recent origin and are the result of pressure from the cities for a share in the motor-vehicle tax revenues, but some of them are of long standing. While the list which follows may not be quite complete, it is believed to include all of the more important instances.

Alabama: 20 per cent of the net receipts from State motor-vehicle licenses collected within municipal limits is returned to the municipality where collected.

California: 50 per cent of the receipts from State motor-vehicle and gasoline taxes is returned to the counties. The city of San Francisco, which is also the county of San Francisco, and apparently also the city of Los Angeles, through the county of Los Angeles, share in the State motor-vehicle revenues through the apportionment to the counties.

Colorado: 50 per cent of the motor vehicle license fees and the gasoline tax receipts collected within each county is returned to the county. Denver city and Denver County are coterminous, and Denver city therefore receives 50 per cent of the State motor-vehicle tax revenues collected within its limits.

Maryland: The city of Baltimore, which is a seperate unit in the organization of the State, receives from the State 20 per cent of the State motor-vehicle revenues.

New York: New York City, as a unit in the county organization of the State, receives 25 per cent of the State motor vehicle registration fees collected within its limits.

Ohio: Cities receive 50 per cent of the State motor-vehicle license fees collected within their limits and 30 per cent of the gasoline tax receipts.

Oklahoma: 90 per cent of the State motor license fee receipts is returned to the county where collected, and 25 per cent of the county's share is returned to the cities and towns within the county.

Pennsylvania: Philadelphia, which is coterminous with Philadelphia County, received in 1925 a grant of $250,000 from the State motor license fee receipts.

Wisconsin: The State makes grants to cities out of its highway fund as follows: (*a*) City streets connecting portions of the State highway system, $300 to $500 per mile, depending upon the classification of the streets; (*b*) other city streets, cities with population over 10,000, $100 to $200 per mile of street, the amount per mile increasing with the population.

OTHER FORMS OF AID TO CITIES

In a few instances State aid is given to large cities for highway purposes in other ways than by grants or by refunds of a fraction of the motor vehicle tax receipts. In Alabama, Iowa, and Kansas the State shares in the cost of construction of highways within city limits which connect the street system with the State highway system. In Washington the State pays the cost of maintenance of State highways within city limits. In other States, as, for instance, California for one State route passing through Los Angeles, and Louisiana for two State routes passing through New Orleans, the State in exceptional cases contributes to the cost of specified State routes passing through or on the outskirts of large cities. As a general rule, subject only to occasional exceptions under special circumstances, all other States require the larger cities and towns to finance from their own municipal funds the pavements which are connecting links in the State highway systems.

In general, also, the counties make no contribution out of county tax revenues to the financing of the streets of large cities, though the situation is complicated in a number of instances by the merging of the county with the municipal governmental organization. Three exceptions to this general rule have been found, however. In Arkansas and Florida the counties turn over to the cities within their limits for street use part of the proceeds of the county road taxes. In the State of Washington 50 per cent of the cost of arterial streets in excess of assessments against neighboring property is borne by the counties or the districts with which such streets connect. Nebraska presents an exceptional instance of the reverse character. Not only do the State and the counties make no contribution to the financing of the streets of the larger towns, but the cities of Omaha and Lincoln are required to contribute out of city funds 50 per cent of the cost of construction of outlet highways outside their limits but adjacent thereto.

These various grants from State and county funds in aid of urban highway finance follow no uniform rule, and there is no evidence which indicates that the basis of apportionment of the State and county funds between urban and rural purposes has in any instance been determined by consideration of the respective ratios of urban and rural use of the highways and urban and rural contributions through taxation to their support. In a number of the instances cited cities share in the State motor vehicle tax revenues only because of the accidental fact that the municipal and county organizations have been merged or are coterminous. These grants are not very important even in the aggregate, an examination of the detailed evidence indicating that they did not exceed $20,000,000

(Continued on page 240.)

COMMON-CARRIER TRUCK FEES AND TAXES

Reported by HENRY R. TRUMBOWER, Economist, United States Bureau of Public Roads

THE special character of business carried on by common-carrier motor trucks is recognized by a great many of the States in the levying of a special license or registration fee upon vehicles engaged in this kind of operation which is considerably higher than the fees charged for the operation of a motor truck by a private operator. At the close of 1925 there were 25 States which charged these extra fees.[1] Certain of these States have also in effect laws which provide that such common-carrier motor trucks may not be operated over regular routes and according to definite schedules unless certificates of public convenience and necessity are first obtained from the public service commission or other regulating agency of the State.

There are also several States which place common-carrier motor trucks under the jurisdiction of their respective public service commissions which do not require a higher license fee of the common-carrier vehicles than they do of trucks operated as private carriers. The States which have in effect public convenience and necessity laws applicable to common-carrier trucks are 27 in number.[2] The States of Arkansas, Idaho, and Mississippi charge special license fees for common-carrier motor trucks, but do not regulate the movement and operation of such vehicles by requiring their operators to obtain certificates of public convenience and necessity before such operations can be engaged in by their owners.

In order to make a comparison of the license fees charged in the 25 States in which special fees are charged the license-fee schedule of each State has been applied to a uniform type of truck conforming to specifications as follows: Pneumatic tires; weight, 7,000 pounds; capacity, 6,000 pounds; 30 horsepower; value, $4,500; annual gross receipts, $12,000; total annual mileage, 20,000 miles; total tire width, 20 inches; total annual gasoline consumption, 4,000 gallons. Certain of these specifications or a combination of them are necessary in order to arrive at the amount of license fees and taxes which carriers of this type have to pay in all States. In Table 1 the license fees which a common-carrier motor truck of these given specifications has to pay are shown in relation to the license fees which in the same States the same type of truck would have to pay if not operated as a common carrier. The difference between the two rates, for lack of a better name, is called the franchise fee.

If one such truck were registered and licensed as a common carrier in each of the 25 States the average fee would be $276.34; if the same truck was licensed as a private carrier the average fee would be only $64.03. The fact that the truck is operated as a common carrier under the assumptions made herein necessitates the payment of $212.31 on the average over and above the fees charged the oridinary truck operated by a business or manufacturing concern but not engaged as a common carrier. This difference may be regarded as a payment for a certain privilege. In all of the 25 States except Arkansas, Idaho, and Mississippi the owners of the trucks operated as common carriers must first obtain permits or certificates of public convenience and necessity, which protect the holders from unwarranted and uneconomical competition on the part of other truck operators who may desire to run their trucks over the same route or routes. On the other hand, the holder of such a certificate finds that his service and rates are regulated by the State in the same manner as rail carriers and public utility corporations.

TABLE 1.—*License fees charged by 25 States for a 3-ton truck of the given specifications when used as a common carrier and a private carrier*

State	License fee as a common carrier	License fee as a private carrier	Franchise fee
Arizona	$120.00	$15.00	$105.00
Arkansas	187.50	125.00	62.50
California	498.00	18.00	480.00
Idaho	600.00	65.00	535.00
Illinois	205.00	75.00	130.00
Iowa	425.00	100.00	325.00
Kansas	185.00	45.00	140.00
Maryland	433.33	9.60	423.73
Michigan	157.50	87.50	70.00
Minnesota	450.00	108.00	342.00
Mississippi	123.75	82.50	41.25
Montana	47.50	37.50	10.00
Nevada	480.00	39.00	441.00
New Mexico	70.00	30.00	40.00
North Carolina	720.00	200.00	520.00
North Dakota	132.00	82.00	50.00
Ohio	140.00	70.00	70.00
Oklahoma	100.00	60.00	40.00
Oregon	45.00	35.00	10.00
South Carolina	150.00	60.00	90.00
South Dakota	360.00	75.00	285.00
Utah	400.00	40.00	360.00
Virginia	520.00	60.00	460.00
Washington	209.00	25.50	183.50
West Virginia	150.00	56.25	93.75
Average	276.34	64.03	212.31

THE BASIS OF THE EXTRA CHARGE

From the discussions in legislative assemblies when proposals are made to charge higher fees for common-carrier motor trucks and passenger vehicles it appears that the theory behind this additional charge is not so much that an extra payment should be made for the so-called "monopoly privilege," but that extra charges should be made in order that the State and the public may receive extra compensation for the use of the highway. In most of these license-fee schedules there is found a mixture of the "privilege" theory and the "use of the highway" theory, with the latter theory predominating. There are two aspects of this theory, one of which may be called the compensatory aspect and the other the business aspect, which are closely allied.

The State takes the position, in the first place, that in licensing two trucks of similar size and capacity the one engaged as a common carrier should be obliged to pay a higher license fee than the private carrier because in all probability the mileage of the common-carrier vehicle will be much the greater and that the operation is in most cases a steady one every day of the year, irrespective of road and weather conditions. Such operation is calculated to result in greater wear and tear on the road, according to the view held by the legislators, involving higher maintenance costs and requiring in some cases a more costly road construction in the first place. The higher fee is intended to cover items of this nature.

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[1] Arizona, Arkansas, California, Idaho, Illinois, Iowa, Kansas, Maryland, Michigan, Minnesota, Mississippi, Montana, Nevada, New Mexico, North Carolina, North Dakota, Ohio, Oklahoma, Oregon, South Carolina, South Dakota, Utah, Virginia, Washington, and West Virginia.

[2] Arizona, California, Colorado, Illinois, Indiana, Iowa, Kansas, Maryland, Michigan, Minnesota, Montana, Nebraska, Nevada, North Carolina, North Dakota, Ohio, Oklahoma, Oregon, Pennsylvania, South Carolina, South Dakota, Utah, Virginia, Vermont, Washington, West Virginia, and Wyoming.

The business aspect is quite different. The fact that an individual or a company puts upon the public highway a motor truck not engaged in transporting the products or goods of the owner but in carrying goods offered for transportation by other business men or the public puts the owner and the truck in a special class; the public highway in such a case becomes an important element in a new business enterprise, and profits are sought by motor-truck operators in this sort of enterprise. There arises, therefore, in the minds of legislators the feeling that where operators use the highways as a part of their business enterprises an extra fee should be charged. Furthermore, in considering this type of highway transportation from the business point of view the legislator is urged, at least by those interested in rail transportation, that these extra license fees and taxes levied against common-carrier motor vehicles should be fixed at a point which will equalize competitive costs.

The common-carrier license fees differ widely in the several States, as will be noted, and the differences between the common-carrier fees and the private-carrier fees are still more pronounced. The common-carrier fees range from $45 in Oregon to $720 in North Carolina, and between these limits the various fees are classified as shown in Table 2. Slightly over half of the States, as will be seen from this tabulation, charge less than $200 as a license fee for this type of carrier; approximately a fourth charge between $400 and $500. The two which in the table are shown to receive more than $600 are States which charge certain percentages of the gross earnings in lieu of a regular license fee. The $12,000 gross earnings assumed in this hypothetical case bring the taxes up to these high amounts. The earnings assumed, however, averaging $40 a day, are believed to be reasonable and normal for this size of truck.

Table 2.—*Classification of common-carrier motor-truck license fees in 25 States*

License fee	Number of States	Per cent	License fee	Number of States	Per cent
Less than $100	4	16	$500 to $599	1	4
$100 to $199	9	36	$600 to $699	1	4
$200 to $299	2	8	$700 to $799	1	4
$300 to $399	1	4			
$400 to $499	6	24	Total	25	100

WIDE DIFFERENCE IN FRANCHISE FEES

The franchise fees vary from $10 in Montana and Oregon to $535 in Idaho, their classification between these limits being as set forth in Table 3.

Table 3.—*Classification of franchise fees for common-carrier motor trucks in 25 States*

Franchise fees	Number of States	Per cent	Franchise fees	Number of States	Per cent
Less than $100	11	44	$400 to $499	4	16
$100 to $199	4	16	$500 to $599	2	8
$200 to $299	1	4			
$300 to $399	3	12	Total	25	100

It is significant that the average of the franchise fees is over three times as great as the average fee paid by this type of truck not registered as a common carrier. Although in 44 per cent of the States it is less than $100, the high additional fees in a number of the other States bring the average up to over $200.

In order to determine to what extent there is a trend towards uniformity and what motives predominate in drawing up license-fee schedules, it is of interest to analyze and classify the various methods followed in arriving at the exact fees to be charged. Table 4 has, therefore, been prepared to show the various bases used in determining the license fees for common-carrier and privately operated trucks in the 25 States, and Table 5 to summarize and classify the same information.

Table 4.—*Basis of license fees for common-carrier and privately owned trucks in 25 States*

State	Basis of private truck license	Basis of common-carrier truck license
Arizona	Capacity	Capacity-mile.
Arkansas	do	Capacity.
California	Flat rate plus weight	Flat rate plus weight plus gross receipts.
Idaho	Capacity	Gross receipts.
Illinois	Gross weight	Gross weight.
Iowa	Capacity	Capacity-mile.
Kansas	do	Capacity.
Maryland	Horsepower	Capacity-mile.
Michigan	Weight	Weight.
Minnesota	Value	Value.
Mississippi	Capacity	Capacity.
Montana	do	Flat rate plus capacity.
Nevada	Gross weight	Gross receipts.
New Mexico	Flat rate plus capacity	Flat rate plus capacity.
North Carolina	Capacity	Gross receipts.
North Dakota	Value plus weight plus horsepower plus capacity.	Value plus weight plus horsepower plus capacity.
Ohio	Weight	Capacity.
Oklahoma	Capacity	Mileage plus capacity.
Oregon	Tire width	Tire width.
South Carolina	Capacity	Gross weight plus capacity-mile.
South Dakota	do	Gross receipts.
Utah	do	Capacity-mile.
Virginia	do	Gross weight-mile.
Washington	Weight	Gross weight plus gross receipts.
West Virginia	Capacity	Capacity-mile.

Table 5.—*Classification of license fee basis for common-carrier and privately owned trucks in 25 States*

Basis	Private trucks, number of States	Common-carrier trucks, number of States
Capacity	14	4
Weight	3	1
Gross weight	2	1
Flat rate plus weight	1	
Horsepower	1	
Value	1	1
Flat rate plus capacity	1	2
Value plus weight plus horsepower plus capacity	1	1
Tire width	1	1
Gross receipts		4
Capacity-mile		5
Flat rate plus weight plus gross receipts		1
Mileage plus capacity		1
Gross weight plus capacity-mile		1
Gross weight-mile		1
Gross weight plus gross receipts		1
Total	25	25

In 22 of the 25 States listed weight in some form or other constitutes the chief basis for determining the amount of the ordinary license fees; in 14 States the rated carrying capacity of the truck is used as a basis; in 3 the unladen weight of the truck; in 2 the gross weight of the truck; and in the others there are combinations of weight or capacity with a flat charge and with horsepower. Only 3 States use as a basis some factor other than weight (or capacity). One of these uses horsepower, another the value of the truck, and the third tire width.

HIGHWAY USE A POPULAR BASIS FOR COMMON-CARRIER FEES

When we come to a study of the basis used in determining the license fees for common-carrier motor trucks it is seen that the use which the truck makes of the highway becomes one of the most important considerations. The license fees charged for the private trucks are in no way affected by the extent to which the highway is used. A truck of a given size or capacity pays in the several States the same respective license fees whether it is operated during a year's period 100 miles over the public highways or 20,000 miles, so long as it is used privately. But in the common-carrier rates, on the other hand, there is a decided shift in the basis of the fees, tending from the fixed fee regardless of the use made of the highways or the amount of business done to a fee which reflects to a large degree the amount of operation over the highway and the amount of business transacted. Fourteen of the 25 States have introduced this factor in some form, measured variously by gross receipts, mileage, capacity-mile, or weight-mile. The gasoline tax is, of course, another way in which recognition is given to this factor of highway use; and this tax is paid by the privately owned trucks as well as by the common carriers. But, presumably, it is paid in greater amount by the latter because of their more extensive operation.

In the following analysis the basis of the common-carrier fees in these 25 States are examined in detail in order to develop more clearly the principles followed in the taxation of vehicles of the special class.

Fees based on truck capacity.—Of the four States, Arkansas, Kansas, Mississippi, and Ohio, in which license fees for common carriers are based on the capacity of the vehicle, two, Arkansas and Mississippi, arbitrarily add 50 per cent to the rates charged for private trucks. Kansas charges the common-carrier truck the regular license fee for trucks which is based on capacity, and then adds to this a flat fee varying with the capacity of the truck. In Ohio the private truck is charged a fee the amount of which depends upon its weight. The common-carrier truck, on the other hand, pays a fee which is based on its carrying capacity. Common-carrier trucks that are not operated over regular routes and between fixed termini pay from 30 to 40 per cent less than trucks that are scheduled to operate over regular routes. This differential is explainable in part upon the assumption that the truck that does not operate over a regular route will not make as much use of the highways as a regular truck, and in part by reason of the special privileges granted to the truck engaged in regular service over a definite route.

Fees based on truck weight.—Michigan is the only State in which the fee depends solely upon the weight of the truck. The fees for private trucks are determined upon the same basis, but in the case of the common carrier an extra charge for each 100 pounds is added to the regular license fees.

Fees based on gross weight of truck.—In Illinois private trucks are classified according to gross weight (weight of the truck plus its rated capacity) and the fees are fixed accordingly. A common-carrier truck has to pay, in addition to these regular registration fees, a special fee of $1 per 100 pounds gross weight.

Fees based on the value of the truck.—In only one State, Minnesota, is value made the basis upon which license fees are charged for all types of motor vehicles, passenger cars and trucks alike. The rate for private trucks is 2.4 per cent of their value (the value being the manufacturer's price less a fixed rate of depreciation per year), whereas a truck engaged in the general transportation business over a regular route is obliged to pay a license fee of 10 per cent of its value.

Fees based on flat rate plus capacity.—Montana varies its schedule of license fees for private trucks according to capacity. The same rates are charged for common carriers, with an additional fee amounting to not more than $10. In New Mexico, the only other State which uses this basis, a flat sum is charged, and to this is added a certain rate per 100 pounds of carrying capacity. In the case of the common-carrier truck both the flat rate and the rate per 100 pounds capacity are increased by about 50 per cent.

Fees based on value plus weight plus horsepower plus capacity.—North Dakota employs the four factors of value, weight, horsepower, and capacity in arriving at the license fee for a private truck. For a common carrier the capacity rates are practically doubled, and in addition a special fee, based upon capacity, must be paid to the railroad commission, the State agency which has jurisdiction and control over common-carrier motor truck operations.

Fees based on tire width.—Oregon bases its license fees for private motor trucks upon total tire width. Trucks which engage in common-carrier service pay 50 cents per inch of tire width more than the other trucks, which is an increase of about 25 per cent.

STATES IN WHICH HIGHWAY USE IS THE BASIS

Fees based on gross receipts.—Four States use the gross receipts from the operation as the basis for determining the amount of license fees or taxes which a common-carrier motor truck should pay. It is held that the amount of receipts is a reasonably fair index of the amount of business done, and in turn of the use made of the highways by the vehicle producing the earnings. The four States following this method are Idaho, Nevada, North Carolina, and South Dakota, and the charges range from 3 per cent to 6 per cent of the gross receipts. South Dakota collects 3 per cent, Nevada 4 per cent, Idaho 5 per cent, and North Carolina 6 per cent. In some cases the payments are made monthly and in other cases quarterly.

Fees based on the capacity-mile.—In charging license fees which are based upon the number of miles operated multiplied by the number of tons of carrying capacity, an attempt is made to make the contribution of common-carrier motor trucks depend directly upon the use which is made of the highways. Tonnage and mileage are the two best indices. Actual tonnage would be still better, but in order that the law may be practically administered it is necessary to use capacity instead. In most cases the collection is made in advance at the time the license is issued. The carrying capacity of the truck and the mileage over which it is proposed to operate it are indicated on the application, and from these data the capacity-mileage for the year is computed.

Five States have adopted this basis. The fees range from one-sixth to two-thirds of a cent per ton-mile. Arizona charges one-fifth cent, Iowa one-half cent per ton-mile. Maryland grades her ton-mile rate from one-sixth to one-third and one-half cent per ton-mile, depending upon the weight of the truck and the type of tire equipment. Utah instead of classifying the truck classifies her highways. Common-carrier trucks operating over hard-surfaced roads pay a rate of two-third of a cent per ton-mile, while those which operate over roads that are not hard surfaced pay at the rate of only one-fourth cent per ton-mile. This classification involves two well-recognized principles of highway finance. The first, which is axiomatic, is that the more a State spends on highway improvement the more revenue it must have to meet such expenditures; and the second is that the greater the degree of highway improvement the more will motor-vehicle operators be able to afford in the way of license fees or taxes on account of the reduction in the costs of operating the motor vehicle. In this case the license fee charged for operating over hard surfaced roads is more than double that charged for operating over other types of roads, and there is slight doubt that the operators gladly pay the additional fees in order to have the benefit of the better roads. West Virginia charges a rate ranging from one-fourth to one-third cent per ton-mile, depending upon the weight of the vehicle used.

Fees based on flat rate plus weight plus gross receipts.—California is the only State in this group. The private carrier is charged a fee based on the weight of the truck plus a flat fee of $3. Common carriers have to pay this same fee plus 4 per cent of the gross receipts.

Fees based on mileage plus capacity.—In Oklahoma the private motor truck is charged a license fee depending on the capacity of the truck. A common carrier pays this same fee, and in addition is obliged to pay a fee of one-fifth cent per mile on the distance traveled.

Fees based on gross weight-mile.—The private motor-truck fee in Virginia is based on capacity. The fee for a common carrier, on the other hand, is determined by multiplying the gross weight of the vehicle by the number of miles traveled times one-fifth cent for the smaller trucks, two-fifths cent for larger trucks, and three-fifths cent for the trucks of the heaviest class.

Fees based on gross weight plus gross receipts.—Washington is the only State using this method. Private trucks pay fees which are based on weight. Common-carrier trucks are charged on the basis of gross weight, and in addition an amount not to exceed 1 per cent of the gross receipts. The part of the fee which is based on the gross receipts is credited to the funds of the public service commission and is intended to cover the costs incurred by that body in the regulation and supervision of common-carrier motor vehicles.

GASOLINE TAXES OF THE 25 STATES

These license fees are supplemented in every one of the States under consideration, except Illinois, by gasoline taxes which vary from 1 to 5 cents per gallon. It is assumed that the truck which has been used in this study for comparative purposes will consume approximately 4,000 gallons in operating 20,000 miles, and on this basis the gasoline taxes which would have to be paid in the several States, together with the license fees and the total license fees and gasoline taxes, are set forth in Table 6.

TABLE 6.—*Common-carrier motor-truck license fees and gasoline taxes of 25 States*

State	License fee	Gasoline tax		Total license fee and gasoline tax
		Rate per gallon in cents	Amount	
Arizona	$120.00	3	$120.00	$240.00
Arkansas	187.50	4	160.00	347.50
California	498.00	2	80.00	578.00
Idaho	600.00	2	80.00	680.00
Illinois	205.00	-------	---------	205.00
Iowa	425.00	2	80.00	505.00
Kansas	185.00	2	80.00	265.00
Maryland	433.33	2	80.00	513.33
Michigan	157.50	2	80.00	237.50
Minnesota	450.00	2	80.00	530.00
Mississippi	123.75	3	120.00	243.75
Montana	47.50	2	80.00	127.50
Nevada	480.00	4	160.00	640.00
New Mexico	70.00	3	120.00	190.00
North Carolina	720.00	4	160.00	880.00
North Dakota	132.00	1	40.00	172.00
Ohio	140.00	2	80.00	220.00
Oklahoma	100.00	3	120.00	220.00
Oregon	45.00	3	120.00	165.00
South Carolina	150.00	5	200.00	350.00
South Dakota	360.00	3	120.00	480.00
Utah	400.00	3½	140.00	540.00
Virginia	520.00	3	120.00	640.00
Washington	209.00	2	80.00	289.00
West Virginia	150.00	3½	140.00	290.00
Average	276.34	---------	105.60	381.94

The average license fee paid in the 25 States by a common-carrier truck of the assumed specifications is $276.34, the average gasoline tax $105.60, making a total of $381.94 per truck. The total payments range from $127.50, the charge made by Montana, to $880, by North Carolina. In the light of the average the gasoline tax is the smaller part of the total payment, averaging 27 per cent of the total. There are 5 States, however, where the gasoline tax exceeds the license fee; these are Montana, New Mexico, Oklahoma, Oregon, and South Carolina.

In addition to these two types of fees and taxes the common-carrier motor trucks are subject in most States to the general property tax. The amount of that payment depends altogether on the assessed valuation and the tax rate. It should be noted, however, that seven of the States dealt with in this article have relieved the common-carrier motor truck by specific statutory exemptions from the payment of property taxes. These States are Idaho, Iowa, Michigan, Minnesota, North Dakota, Oklahoma, and Oregon. In a number of the States, on the other hand, cities and incorporated towns are permitted to charge special license fees for trucks engaged in common-carrier service and make a practice of doing so. The available data are not sufficient to determine closely what such municipal license fees and general property taxes amount to in the several States, but it is believed that they amount to a fairly substantial yearly payment on the average.

INTERSTATE OPERATION DOUBLY TAXED

Where interstate operations are involved common-carrier motor trucks operating over regular routes between two States are required to take out licenses in each of the States. If the license fees in each of the States involved are measured by a mileage or gross-earnings factor the total registration fee is no greater for the interstate than for intrastate operation. But if the interstate operation occurs between two States each of which has in effect license-fee schedules based upon capacity or weight it is necessary to pay the full license fee in each State. For example, a common-carrier motor truck operating between a point in Minnesota and a point in North Dakota would have to pay the $450 fee for a 3-ton truck with the given specifications in Minnesota, and $132 in North Dakota in addition to the gasoline tax. In all of the States except those which base their fees on ton-mileage or gross receipts it is more expensive to operate common-carrier motor trucks in interstate than in intrastate commerce.

These average license fees and gasoline taxes, which for a common-carrier truck amount to $381.94 are approximately 3.2 per cent of the assumed annual gross receipts of $12,000. How nearly this ratio actually holds true can not be definitely determined on account of the lack of operating and financial reports. According to the assumptions that have been made this type of motor truck under consideration would have to pay in Maryland a license fee and gasoline taxes which would amount to 4.3 per cent of its annual earnings. In the same State eight motor-truck operators reported in 1924 that their taxes and license-fee payments amounted to 5.7 per cent of their gross receipts. In this report the personal property taxes were undoubtedly included, but the taxes concealed in their gasoline costs do not appear as a tax item. Accordingly, it appears that at least in this particular case the total tax burden, including personal property taxes, license fees, and gasoline taxes, amounted to a sum in excess of 6 per cent of the gross earnings.

It may be of interest further to compare these tax payments with the tonnage carried in order to determine the amount of the tax burden on the actual service rendered. Table 7, therefore, shows the total of license fees and gasoline taxes for each State reduced to a ton-mile basis, using the assumptions made throughout this article.

Table 7.—*Tax-burden per ton-mile*

State	License fee and gasoline tax per capacity ton-mile	State	License fee and gasoline tax per capacity ton-mile	State	License fee and gasoline tax per capacity ton-mile
	Mills		*Mills*		*Mills*
Arizona	4.0	Minnesota	8.8	Oregon	2.7
Arkansas	5.8	Mississippi	4.0	South Carolina	5.8
California	9.6	Montana	2.1	South Dakota	8.0
Idaho	11.3	Nevada	10.6	Utah	9.0
Illinois	3.4	New Mexico	3.2	Virginia	10.6
Iowa	8.4	North Carolina	14.7	Washington	4.8
Kansas	4.4	North Dakota	2.9	West Virginia	4.8
Maryland	8.6	Ohio	3.6		
Michigan	3.9	Oklahoma	3.6		6.3

The average tax burden of 6.3 mills per capacity ton-mile is based on the assumption of a full capacity load for every mile operated. For a 50 per cent load factor these fees and taxes would amount to 12.6 mills per ton-mile, and for a 75 per cent load factor they would be 9.4 mills per ton-mile. As investigations have shown that the actual load factor commonly ranges between 50 and 75 per cent, it may be fairly assumed that the latter averages are nearer to the fact than the lower average shown in Table 6.

An examination of license fees and an investigation of tax burdens which the several States are placing upon motor trucks engaged in common-carrier service tends to show that in the past few years many of the States have passed laws requiring the common-carrier truck to pay license fees which on the average are approximately four times the fees charged for privately operated trucks. More and more States have followed this policy of exacting higher fees from the common-carrier vehicle. There appears to be a lack of uniformity as to measures employed to determine the amount of the fee, but there is evidence of a tendency to use means which reflect the use made of the highways furnished for this business by the public and which furnish an index of the operator's ability to pay in that he is supposed to be able to pay a higher fee if his gross earnings are large than if they are small.

URBAN ASPECTS OF HIGHWAY FINANCE

(Continued from page 235.)

in 1923 for all cities in the United States having a population of 30,000 or over.

Small towns, and especially those under 2,500 in population, receive more generous treatment. It is the general rule or the frequent practice in a majority of the States for the State itself to construct or to provide funds for the construction of standard sections of State highways passing through towns not exceeding a specified population; in some cases maintenance also is provided by the State. This is a desirable practice, if for no other reason than to provide a safeguard against serious impairment of the efficiency of State highway systems through the persistence of unpaved or unsatisfactorily maintained sections within the limits of small towns. In a few States, especially in New England, small towns receive aid from the State in financing their general street program, whereas larger cities receive no such aid. This may perhaps be justified on the ground that for very small towns the town limits are not to any appreciable extent traffic boundaries, and common financing with the surrounding rural highways is logical. In States in which the county is an important political unit it would be more desirable that any aid given to small towns for their general street program should be given by the county rather than by the State. Such streets, like local rural roads, render little service to the dwellers in large cities, and they should not be made, through their contributions to the State tax revenues, to contribute to their support.

LANDSLIDES TO BE STUDIED

The United States Bureau of Public Roads has assigned Dr. George E. Ladd, geologist, to a study of the problems involved in landslides in West Virginia and Ohio, with particular reference to the effect of slides on road construction.

A preliminary survey has been completed in the two States. For the future it is planned to make a field study of the geological and topographic conditions involved, and a detailed study of the different types of slide which are about equally divided between side-hill and fill slides. Possibilities of surface drainage will be investigated and field experiments will be conducted. In addition to the field work, laboratory experiments will be made on average samples of slide material from different localities.

EFFICIENCY IN CONCRETE ROAD CONSTRUCTION

A REPORT OF OBSERVATIONS MADE ON GOING PROJECTS BY THE DIVISION OF CONTROL, BUREAU OF PUBLIC ROADS

Reported by J. L. HARRISON, Highway Engineer

PART III.—THE EFFICIENCY OF EQUIPMENT

IN THE previous article the fact was brought out that the cost of hauling material increases rapidly with the distance it is hauled. This in turn increases the cost of the paving. It is, therefore, advisable, other things being equal, to keep the haul from the material yards to the mixer as low as possible. But even when appropriate sites for material yards are so favorably located as to make possible a series of short hauls for the entire project, the advantage of these natural conditions may be largely lost to the contractor unless his material-handling plant is so constructed that it can be moved from one site to another expeditiously and at small expense.

If the element of mobility is ignored, the problem of moving from one material-handling yard to the next is a serious one, not so much because of the increase in the direct cost as because of the time involved. When operations must be closed down even for a day or two some of the men are almost certain to leave. Then when operations are resumed men who are not familiar with the work must be employed and trained before the organization will run smoothly. The result is a series of expensive intangible losses very hard to calculate. To avoid such losses every unit in the material-handling plant should be so designed that it can be moved between quitting time at night and the resumption of work next morning.

The probability that the crew will be more or less demoralized by closing down the work is so great that contractors often keep their men employed during periods when the material-yard equipment is being moved, even though the work done is valueless from the production standpoint. This practice generates tangible losses equal to the amount paid for the performance of useless work, but these losses are accepted by the contractors as preferable to the intangible losses faced when trained men leave. As such operations may run for two or three days, the importance of the losses incident to any slowness in changing the material-yard set-up and the real value of high mobility are apparent. Rollers, tractors, trucks, etc., are highly mobile. They can be transferred to a new base of operations cheaply and within a short time. Modern mixers, cranes, and other items of heavy equipment have some degree of mobility, but manufacturers could well give more consideration to this feature in design. There is a good deal of complaint among contractors that when such equipment must be moved any considerable distance under its own power there is a tendency for bearings and clutches to overheat, with the result that frequent long stops have to be made to keep them from burning out. This condition can be and should be so fully corrected that mixers and crawler-type cranes can be moved expeditiously and under their own power any reasonable distance, even over unimproved roads. The exact speed which they should be able to maintain is of less importance. Two or three miles an hour would be fast enough. But they should be able to accomplish these trips without the necessity of making long and repeated stops to cool bearings and clutches.

Bins, hoppers, etc., even many of the best of those manufactured for use on concrete paving jobs, seem to have been designed with little thought of portability. They should be built in rigid sections and provided with holes or handles at points which can be reached by the crane, so that by removing a few pins or a few bolts each section can be lifted down and placed on a truck. There is no reason why the steel hoppers used for sand and coarse aggregate should not be so designed that they can be taken down in a few hours, hauled to the new location, and erected in about the same length of time. Detail plans for both erection and dismantling, as well as instructions governing the performance of these operations, should be furnished with all equipment of this kind.

THE PROPER USE OF THE STOCK PILE

The most commonly accepted practice in handling concrete aggregates is the use of bins and hoppers. This, however, is not the only method applicable to this work. It may therefore be well to refer again to the fact that present-day manufacturing enterprises—and laying a concrete pavement is typically a manufacturing enterprise—do not depend so largely on the accumulation of heavy stocks of materials as they do on properly adjusted rates of delivery of the necessary materials. The stocks serve merely to absorb temporary excesses in the rate of delivery and are drawn upon only when there is a deficiency in the rate of delivery. If, in laying concrete pavements, the rate of rail delivery could be maintained in exact harmony with the rate of use at the mixer, no material would need to go to the stock piles. But this is, of course, a condition impossible of attainment on a job of any length, since weather conditions and other unpreventable causes hinder the attainment of uniformity in the rate of output, even though the production of the mixer while it is at work be maintained at a uniform rate. In practice, therefore, there are periods when some material must go to the stock piles and others when materials must be withdrawn. The closer the approach to a direct transfer of materials from cars to job the greater, of course, will be the economy of operation. A direct transfer involves handling the materials but once. Passing the materials through the stock piles involves handling them twice, and where the piles become very large a third handling may be necessary. In the latter case, if the plant is being worked to capacity, it may be necessary to do overtime or night work in moving the materials, thus adding to the extra handling still another expense.

On by far the larger number of concrete paving jobs the materials are delivered at the material yards by rail. The handling equipment should, therefore, be so designed that it will unload railroad cars efficiently. But as there are also jobs where the materials are secured from local quarries, sand pits, gravel beds, etc., the equipment should also be adaptable to such uses as well as to moving stock-piled materials.

There are four general systems of handling materials used in concrete road construction, in which the operations are as follows:

1. Cars to loading bins or stock piles by crane; bins to trucks or industrial railway cars by gravity.
2. Cars to pit by gravity; pit to stock pile by belt conveyor; stock pile to trucks or cars by elevating bucket conveyors, often known as loading machines.
3. Cranes or belt conveyors to stock piles over tunnels; stock piles to cars, occasionally to trucks, by gravity.
4. Cars to wagons or trucks by hand shoveling; wagons or trucks to job where materials are piled along subgrade; piles to mixer in wheelbarrows.

Of course there are various modifications of all of these methods, and occasionally a new idea is encountered. In general, however, the new methods fail to produce the desired results and are soon abandoned; and much the same may be said of systems 3 and 4, as described above. The latter of these was at one time the all but universal practice; it is now almost entirely out of use. Hand methods are seldom advantageous, even when labor is cheap, for probably the most speculative element in any enterprise of this sort is the output which can be obtained from the labor employed. To reduce the amount of labor required, and particularly to introduce elements which tend to control the amount of work done per man per day, reduces the importance of the speculation involved. The inaccuracy with which the batch is measured has also served to hasten the abandonment of this system of handling materials.

Somewhat the same conditions have all but forced the loading tunnel out of existence. At best the batch is poorly measured by this system; and all things considered, though on the face it is cheap, it has never been very satisfactory.

THE MODERN HANDLING SYSTEMS

Two really modern systems remain. Both work well and can be successfully operated. The scheme of passing material to pits by gravity, to stock piles by belt conveyer, and through loading machines to the trucks is probably somewhat more mobile. It has the disadvantage, however, of generally using four machines to do the work of the crane and the bins, and as a result the number of men required for successful operation is larger. Some advantage accrues to this system from the fact that where the sand is obtained from one point and the coarse aggregate from another the equipment can be readily divided. The percentage of jobs where this condition prevails, however, is small, and as the pay roll is relatively large it does not now, in spite of the high mobility of the machines involved, seem likely to displace the more general method of handling aggregate, the crane and loading bin.

In the performance of any operation efficiency is attained only when the end is accomplished with the least possible work and when the work that is done is done expeditiously and accurately. Judged by this standard the crane and the loading bin have shown their advantage. In a system involving the constant, steady delivery of material the crane has proved to be a dependable piece of equipment. The standard set-up is simple. The hoppers are set at such a distance from the track that the crane, which should be of the crawler type with a ¾-yard bucket, can work freely between the hoppers and the cars. The distance between stock piles should be such that two cars, one of sand and one of gravel, may be reached. The crane will then unload either into the hoppers or onto the stock piles, this being governed by the number of cars to be handled and the requirements of the mixer.

A good ¾-cubic yard crane in the hands of a skillful operator will move a bucket every half minute from a freight car into the hoppers, and will operate at a slightly higher rate from the stock piles into the hoppers, as indicated by the actual stop-watch readings presented in Table 1. When handling sand the loads will average close to the rated capacity except in very dry sand, which will run out badly if the bucket does not close tight. When handling crushed stone or gravel the loads will average about 0.6 cubic yard. The average load obtained while working from a freight car is less, about 0.6 cubic yard for sand and seldom an average over 0.5 cubic yard for stone. This difference is due principally to the difficulty of getting a full bucket in a nearly empty car. A good operator, however, can take a cubic yard of gravel or stone out of a freight car every minute, and a little more than that amount of sand. With good operation, therefore, the output should be about 60 cubic yards per hour; but the daily rate will seldom reach the hourly rate multiplied by the number of hours. One reason is the delay caused by shifting cars. Another is the fact that if a car of sand and a car of stone are being unloaded into the hoppers it is obvious that the car of stone will be emptied first. If the cars have not been spotted in the proper order it will then be necessary either to get stone from the stock pile until the sand car is emptied or else to empty the sand car onto the stock pile before new cars are brought up. Of course it is desirable to have the cars spotted by the railroad so that as a general practice the string waiting to be unloaded is made up alternately of two cars of gravel or stone and one of sand, but it is not always possible to get train crews to exercise this care in setting off and arranging cars.

Table 1.—*Stop-watch readings of time required to handle material by crane from stock piles and cars to batcher plants*

Time required per load from stock pile to batcher plant				Time required per load from cars to batcher plant			
Load No.	Time	Load No.	Time	Load No.	Time	Load No.	Time
	Seconds		*Seconds*		*Seconds*		*Seconds*
1	20	15	26	1	27	17	25
2	20	16	37	2	29	18	28
3	20	17	20	3	28	19	30
4	45	18	23	4	29	20	28
5	28	19	19	5	28	21	30
6	23	20	25	6	34	22	40
7	29	21	31	7	39	23	30
8	18	22	25	8	33	24	29
9	25	23	24	9	28	25	29
10	36	24	30	10	27	26	26
11	21	25	33	11	28	27	28
12	28	26	30	12	26	28	30
13	23	27	27	13	29	29	29
14	26	28	30	14	25	30	29
				15	34	31	33
				16	30	32	32
Average time per load			26. 5	Average time per load			30

The capacity of the crane as a tool for unloading cars is of importance only when for some cause too many have been delivered at one time and it is necessary to get them unloaded in order to avoid demurrage. The vitally important question is whether or not the crane can keep up with the demands of the mixer. It has been stated that the mixer, when working at full efficiency, can turn out 48 batches of concrete an hour. The mix may be as lean as 1:2:4, which requires approximately 10 cubic feet of sand and 20 cubic feet of coarse aggregate per batch. This is a maximum demand of 480 cubic feet, or about 18 cubic yards of sand and 960 cubic feet or about 36 cubic yards of coarse aggregate, a total of slightly over 53 cubic yards an hour. From this it will be clear that the crane is able to keep up with the mixer even when it is working from the cars to the hopper and the mixer is operating at full efficiency, but that it has rather a narrow margin to work on. It is also clear that if manufacturers develop a 6 or 7 bag mixer which can turn out as many batches an hour as the modern 5-bag mixer, a heavier crane with a larger bucket will be needed to keep up with the mixer when it is working at full efficiency.

WINTER STOCK PILING EXPENSIVE

In this connection there is some light that can be thrown upon the practice of stock piling aggregate during the winter, which is so often recommended. By the crane and bin method winter stock piling can be done readily enough wherever space is available. The bucket will take either stone or sand out of the freight cars even though it has been somewhat consolidated by freezing, a more common winter condition than many suppose. The difficulty with this system is its cost. Rail delivery in general is so dependable that if shipments at the source of supply are made according to schedule and the mixer is operating with reasonable efficiency upwards of 75 per cent of the aggregate will go from the cars direct to the job. The other 25 per cent or less will go to the stock piles and then to the hoppers. This generates an average handling of materials approximating 125 per cent. But because few jobs maintain full mixer efficiency this, as a rule, is easily within the capacity of the crane. But when materials are stock piled during the winter there is a 100 per cent unloading onto the stock piles, which then become so large that much of the material must be shifted once, sometimes twice, to get it to a point from which it can be placed in the loading hoppers.

Winter stock piling, therefore, not only increases the cost of handling by the amount of the direct cost of unloading, together with such carrying charges as interest, loss of material (and stock-pile losses are considerable), etc., but also increases the subsequent cost of operation by reason of the fact that if production is well maintained the crane can not shift material and at the same time keep the hoppers filled. The result is that overtime operation must be resorted to in order to keep up the supply of material within reach of the hoppers. In view of these costs and the present outstanding dependability of rail delivery, material concessions in the price of aggregate or in the freight rate for winter delivery must be obtained to justify winter stock piling.

The stock piling of aggregate at convenient points along the road, which is sometimes advocated, is also expensive. In the first place, it increases the truck time required to move the material to its place in the road. Thus, assuming the use of single-batch trucks and the most favorable placement of stock piles, the time required per load for the movement of materials to the stock piles may be represented by the formula:

$$T \text{ (time per trip in minutes)} = 8d_1 + 3, \qquad (1)$$

in which d_1 is the distance in miles from the unloading point to the stock pile and the constant 3 is the time, in minutes, required for loading (lower in this case than in standard operation because cement is not handled with the aggregate).

For the subsequent delivery of the same materials to the mixer the time will be represented by the standard formula:

$$T = 8d_2 + 4, \qquad (2)$$

in which d_2 is the distance from the stock pile to the mixer. The total time required to deliver a batch to the mixer will, therefore, be:

$$T = 8(d_1 + d_2) + 7. \qquad (3)$$

As $(d_1 + d_2)$ is not likely to be less than d, the direct-haul distance from the unloading point, and is often greater, the time as determined from equation 3 is not likely to be less than $8d + 7$, whereas the equation for the time required on direct haul from the unloading point to the mixer[1] is

$$T = 8d + 4. \qquad (4)$$

In some instances the opportunity to make fuller use of a limited truck supply, particularly under conditions involving unusually long haul, may be such as to justify stock piling along the road in spite of the extra hauling cost which this analysis shows to be involved, but as a general practice stock piling in this way can not be defended.

In addition to the increased hauling cost this method has the same disadvantages as stock piling at the unloading point—high material losses and high handling costs. Nor can these ordinarily be offset by any gains at other points. It is sometimes claimed that trucks can be loaded more heavily where materials are stock piled, but as they are now badly overloaded when hauling standard batches this argument is hardly to be countenanced.

MODERN STEEL BINS PREFERRED TO HOMEMADE EQUIPMENT

For handling materials some sort of bin or hopper must be used. The up-to-date contractor has very generally adopted some one of the better types of steel bins. These can be moved from job to job, and, as they last a relatively long time, become a part of the contractor's regular equipment. The homemade bin is going out of use because the measuring devices which may be had with the steel bins are faster, more reliable, and much more accurate. The quicker the operation of the measuring device on the loading bin the less truck time is involved per trip. First-class modern measuring devices on good steel hoppers will measure and discharge a 5-bag batch (sand and gravel) in less than 15 seconds, as indicated by Table 2. As the old-fashioned homemade devices often take a minute or so to load the coarse aggregate and half that time to load the sand, the saving in truck time from the use of modern measuring devices more than offsets their greater first cost.

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[1] See the second of this series of articles, PUBLIC ROADS, vol. 6, No. 10, December, 1925.

TABLE 2.- *Stop-watch readings of time required to load a 5-bag batch of sand and gravel from a modern batcher plant to single-batch trucks.*

Batch No.	Time	Batch No.	Time	Batch No.	Time	Batch No.	Time
	Seconds		*Seconds*		*Seconds*		*Seconds*
1	11	19	16	37	17	55	11
2	22	20	12	38	18	56	10
3	14	21	12	39	10	57	9
4	12	22	14	40	8	58	10
5	10	23	11	41	16	59	14
6	15	24	12	42	17	60	11
7	27	25	20	43	14	61	10
8	6	26	19	44	20	62	12
9	14	27	9	45	16	63	21
10	25	28	38	46	13	64	24
11	17	29	12	47	14	65	13
12	26	30	8	48	8	66	6
13	8	31	8	49	13	67	13
14	13	32	6	50	13	68	25
15	10	33	9	51	16	69	42
16	22	34	12	52	12	70	11
17	10	35	12	53	16	71	12
18	14	36	15	54	20	72	7
Average time required to load sand and gravel							14.5

Accuracy is a vital factor in producing first-class concrete. The right amount of material should be sent out every time. Homemade devices are, in general, definitely inaccurate. The better of the modern hoppers, on the other hand, measure very accurately; and there are now obtainable accurate and thoroughly practical weighing hoppers, and another device which measures the sand under water, a device by which the correct amount of sand and a constant water content are obtained by the same operation.

Of these three systems for measuring aggregate the first, the determination of the batch by measurement, is the fastest. The second, however, is quite fast enough for all practical purposes, as the complete operation of weighing a batch and discharging it is commonly performed in from 40 to 45 seconds. No operation which can be consistently performed in under 1¼ minutes (the standard mixing cycle) need delay the mixer. On the other hand, every operation of this sort does add to the round-trip time of the trucks. But as the weighing can be done between the arrival of trucks, the use of the weighing device does not appreciably affect the trip time of single-batch trucks as compared with their trip time when the measuring device is used, though it does have a slight adverse effect on the trip time where two-batch or larger trucks are used.

The device by which the sand is measured under water is not in common use on highway work. At present it has no particular advantage over the weighing or usual measuring process except at central mixing plants; for, while it perhaps yields a slightly greater accuracy in the determination of the sand content of the mix and a more uniform moisture content in the sand, neither is of much practical value and some rather serious difficulties attend the use of the device under ordinary field conditions. Accuracy in measurement is, of course, desirable, but the accuracy of the weighing process is such that, even with due allowance for variation in the water content of the sand, the error does not often exceed 1 per cent, which is far below the margin of error in other and more important elements in paving work.

A uniform water content in the sand also is highly desirable, but can be of little practical value until the valves in the mixers are so improved that leakage as a common cause of irregularity in the water content is eliminated. Moreover, the high water content in sand which has been measured in this way makes it difficult to handle the cement in the same transportation units, and also increases the lag in discharging the skip at the mixer.

The value of effective modern batch measuring or weighing devices may perhaps be more clearly shown by referring again to the fact that the homemade devices often require as much as a minute for charging a single batch of coarse aggregate and half that time for charging a single batch of sand. With no allowance for the truck time commonly required in getting from the coarse aggregate bin to the sand bin, the time required in loading these materials may, then, reach 1½ minutes per batch, or if 40 batches are poured in an hour, to an hour's truck time per mixer hour. As against this a well-designed modern measuring or weighing device will discharge so quickly that in practice about all that a single-batch truck has to do is to come to a stop and then move off with the load, the next batch being measured or weighed in the interval between the loading of one truck and the arrival of the next. This does not, of course, reduce the time lost to zero, but it comes so close to it that in practice it may take less time to load single-batch trucks from a device of this sort than it does to move them from the coarse aggregate bin to the sand bin of a homemade plant. The saving in truck time which is to be had from the use of a first-class loading device may, then, when operation at the mixer is at a rate as high as 40 batches an hour, be equal to the full time of one single-batch truck. The saving when two-batch trucks are used is quite as important, but does not lend itself as easily to the method of presentation used here. This point should, however, be clear, that savings in truck time obtainable from the use of modern batch measuring or weighing devices are such as to amply justify their cost.

PRESENT METHODS OF CEMENT HANDLING INEFFICIENT

The most inefficient process on concrete paving jobs to-day is handling the cement. This is still all but universally conducted as a manual operation. Three methods are in common use. By the first the cement is brought to the job in separate trucks, where it is piled along the subgrade; by the second the proper number of unopened sacks are placed on top of the loaded material trucks or cars; and by the third method the proper number of sacks are emptied by hand into the trucks or whatever transportation units are used. Whether the sacks are emptied into the trucks at the cement house or at the mixer, or from the piles along the subgrade into the skip, the operation takes about a minute. If the sacks must be emptied on the truck at the mixer the slightest delay in performing this operation, as when a sack gets buried in the sand, delays the mixer. Similarly, when the sacks are dumped into the skip, if a truck is delayed the operation holds up the mixer, for most inspectors require that the cement be dumped upon the aggreate, refusing absolutely to permit dumping the aggregate upon the cement. In either case the loss of time at the mixer amounts to enough—a minute here and there throughout the day—so that contractors can well afford to consider more effective methods of handling this material.

The operations incident to laying a concrete pavement are such that perfection can be secured only as equipment and methods are perfected and men are thoroughly trained. Specifications do not ordinarily demand perfection. In general they are reasonable, and clearly indicate that all that is expected is work of reasonably attainable quality. Under such conditions the real test of a method or practice is not whether it is perfect, but whether in permitting it there is any real danger that the quality standard established by the specifications and maintained in other parts of the work will be jeopardized. A contractor ought to be permitted to handle his work expeditiously and economically in all of its particulars unless it can be shown that to handle it in this way will clearly result in a finished product which is below standard. The only sound method of determining this is to test any proposed practice in the field, and by so doing to ascertain whether it will produce a product measurably equal to that produced by the methods and practices in vogue.

But whether the contractor's view that the cement ought to be treated as part of the batch or the view that it is deserving of separate and very careful treatment prevails, the fact still remains that cement can be more efficiently handled than is customary at the present time. A number of solutions are possible. A belt conveyer can be installed in the cement house at a reasonable cost to elevate the sacks to a point where the cement can be emptied into hoppers to be dropped by gravity into trucks provided, if need be, with special cement compartments. An installation of this sort recently viewed by the writer required only three men. Cement was trucked from the cars to the conveyer by one man, loaded on the conveyer by another, dumped into the hoppers and the hoppers discharged into the trucks by the third. This arrangement handled the cement successfully up to 225 sacks an hour, which was the greatest demand made upon it. Rather close observation of the work done indicated, however, that it could have met without difficulty the full production demand of 240 bags an hour required for 48 batches an hour.

Nor is there any reason to suppose that hoppers equipped with weighing batch boxes can not be readily adapted to this purpose. If this were done some, at least, of the objections to the shipment of cement in bulk would be overcome. Doubtless other mechanical methods of dealing with the cement problem will be developed, but however this may be, the fact that handling cement now commonly requires about twice as many men as are used in handling five to seven times the weight of aggregate indicates something of the opportunity which there is for saving labor at this point in the paving contractor's work.

A development of the cement-handling problem such as was used in discussing batch measuring and weighing devices would show that at present the truck time involved in handling the cement is about a minute where single-batch trucks are used. To cut this in half would save 20 minutes of truck time per hour if the pouring rate is 40 batches an hour. The belt-conveyer system referred to above required three men as against the five to six commonly employed about the cement house. The saving possible from the device described above is, then, the total time of two men, in some cases three, and at least one-third of the time of one truck. This is quite sufficient to cover the operating cost of the equipment involved, depreciation, interest, etc., and leave a profit for the contractor.

Summarizing the points in regard to material handling which have been touched on above one thing in particular deserves emphasis, namely, that if full production is to be secured no one of the loading operations can be so complicated that the time needed in performing it takes over 1¼ minutes without making this the pace setter for the job. A homemade bin that is so poorly constructed that 1½ minutes are required in loading stone cuts production to 40 batches an hour. A cumbersome method of handling cement may have the same result. A crane so poorly operated that it can not get a batch into the loading bin every 1¼ minutes has the same effect. There is here, then, a sound basis for urging that the best of equipment and the best methods be used in the material yards. On the other hand, every minute saved in performing a service operation such as these also increases the efficiency obtainable from the transportation equipment, which of itself is an important item. But when cumbersome methods reach so far that they reduce the output at the mixer regardless of the transportation available, their cost mounts so sharply that, if competition is keen, no contractor using them can hope to remain long in the business.

FORM SETTING AND SUBGRADE PREPARATION

Preparing the subgrade and setting the forms are operations which, in general, do not much interfere with production. Form setting is a manual operation. This is difficult to avoid, as great accuracy is imperative, a satisfactory surface finish being next to impossible of attainment if the forms are carelessly or inaccurately set. Inaccuracy in maintaining the distance between the forms may also cause trouble in the operation of the finishing machine. If the spacing of the forms is wide the wheels bind, if too narrow the finishing machine will run off.

The form setting has been found occasionally to interfere with production because of an inadequate supply of forms, but in such cases the remedy is obvious. Specifications commonly require that forms be left in place 24 hours. It is good practice to set each day the length of form to be used the next. This requires a minimum of twice the daily run, and as a fair margin is always desirable, a form supply of from two and one-half to three times the daily run is apt to prove helpful.

A few years ago finishing the subgrade was almost entirely a manual operation. To-day it is almost as completely a mechanical operation. Unfortunately, however, it has not been standardized. There are, for instance, regions where heavy rolling is an absolute requirement. It matters not at all that the rolling works the subgrade as kneading does putty until it becomes unstable and trucks sink almost to their hubs. The rolling is still insisted upon. There are other regions where heavy rollers are not permitted on the job. Other differences in the treatment of this outstanding important element could be noted, but there is no reason for doing so, as the purpose is merely to suggest that the fact of such wide variations in the treatment of the subgrade indicates a lack of real information concerning what ought to be done to place it in the best condition. To the contractor, however, this difference in specified practices is of more importance, for it is a matter of practical experience with him that the men he has trained to prepare subgrade satisfactorily in one State must be retrained in the next—perhaps even in the next district or the next county. This, of course, slows up operations and increases costs.

But whatever the methods required in finishing the subgrade one modification should be made in dealing with it. It is the common practice to set grade stakes and to finish the subgrade, if this is done some time in advance of surfacing, to the bottom of the concrete paving at the crown. As a result, the whole operation of shaping the subgrade for the concrete slab is one of trenching. The process is expensive, for, while the quantities handled are small, the blade graders used for this purpose carry the cuttings toward the center of the road from which point they must be removed to the shoulders by slip or by fresno. The net result is a triple handling of this material. First, when the grade is built; second, by the blade in cutting it out and throwing it to the center of the road; and third, when it is dragged to the shoulders. It would much simplify this operation if the subgrade were staked to a mean point so calculated that the cuttings from the edge (including the take out for setting the forms) would just make the crown at the center. This practice would lend itself readily to the mechanical methods now in common use in finishing the subgrade and should be in the interest of economy, for, while a little shoulder material would have to be brought in during the final clean up, this could hardly cost as much as it now costs to move this same amount of material three times.

It is appreciated, of course, that the defense of the present practice lies in an assumption that cutting away material as described above leaves a more uniform subgrade condition than could otherwise be attained. If the cutting process were or could be accurately accomplished there would be much merit in this assumption. The fact is, however, that it is not accurately done and in the nature of the case can not be with the tools now available. The blade leaves many high spots and takes out many areas below grade. As a result the only certain results of this practice are a more or less uneven compaction of the surface of the subgrade and a needlessly high cost of operation.

SUBGRADE PRACTICE RECOMMENDED

Where specifications will permit the following practices, the fastest and cheapest method of finishing the subgrade is the one described below, which, except as to item 1, is in use in a number of places in the Middle West.

1. Rough grade to mean elevation as described above. Ordinarily a shrinkage factor is allowed when subgrades are laid down in advance of paving, and as this seldom proves to be correct some additional rough grading is likely to be required.
2. With blade, cut slope for extra edge thickness of pavement, cutting wide enough to provide for setting the forms.
3. Trim out for and set forms by hand.
4. Blade material for crown to approximately its final position in the center of the road.
5. Scarify subgrade thoroughly to at least 2 inches below the finished grade.
6. Cut subgrade to exact shape with standard subgrader pulling the machine at least twice to insure a good job. For this work, set the blades from one-quarter to one-half inch high as experience in the particular soil dictates. Roll lightly with a light (1½ to 3 tons) roller. Pull subgrader after rolling with blades set to correct elevation.
7. Back of mixer drag a modern fine finisher to insure that distortion subsequent to preparation of subgrade is eliminated. If the subgrade is of clay or similar material and has been repacked by the trucks, sprinkle heavily under the mixer and on any high spots to facilitate cutting and if the finisher still rides high have four or five men stand on it while it is being pulled ahead by the mixer.

These operations are about as completely mechanical as they can be made at present, are simple and direct, involve the least possible rehandling of material, and result in a subgrade as perfectly shaped as it is now possible to attain on a commercial scale. The one objection likely to be raised is that the scarifying process produces a loose layer under the pavement. Admittedly this is true. Indeed, the process produces a condition quite different from that secured when heavy rolling is required. Extended observation of the use of heavy rollers, however, leaves the impression that where they are used either the actual cross section only roughly approaches that called for on the plans or, and this is probably of much more importance, the tendency is to secure regularity in the final treatment of the cross section by cutting off high spots and filling low spots with the cuttings which are not thereafter as fully compacted as in their first placement. The result is a succession of hard spots and loose areas—not a satisfactory bed on which to place an expensive surfacing. The method outlined above does not wholly correct this difficulty, but its general effect is to procure a more uniform degree of compaction beneath the entire pavement and to produce a condition wherein the effect of filling areas that are a little low is more nearly in keeping with the general condition of the subgrade.

To the contractor an even and exact finish of the subgrade is an important consideration. If he is required to give at least the depth of pavement called for on the plans, as he should be, he ought to be permitted to so handle his subgrade that he will give no more, providing this is possible without endangering the structural integrity of the payment. It is, of course, equally true that if practices are required which make the cost of filling low places with subgrade materials as great as or greater than the cost of filling them with concrete, then the permission to fill them has no practical value. When the use of a heavy roller is specified for the final compaction of the subgrade, a practical solution of this problem of securing a true surface at exact grade at all points is almost an impossibility. But where the method suggested above is used, no such trouble exists, and the contractor is able to give a full depth of pavement and at the same time avoid the use of any considerable excess of materials, which is an important consideration, as a consistent overrun in quantities—even if it amounts only to 2 or 3 per cent—has an important adverse effect on profit.

TWO-INCH PIPE LINE MUST GO

The absolute dependence of production on an effective material supply has been emphasized throughout this series of articles. The water used in mixing—and in curing as well—is as truly one of the materials as are cement and stone. With the water as with the stone or the sand, methods of handling and means of delivery are vital factors in efficiency. Only one system of handling water is in common use to-day. It is pumped through a pipe line laid along the right of way to the mixer. In the common practice provision is made at regular intervals—generally 200 to 300 feet—for taking water from the pipe line, and a hose connection is used to feed the mixer. These hose connections are also used to obtain water for sprinkling and for any other job requirements. A study of the "lost time" on the job [2] will show that the time lost as a result of trouble with the water supply is rather a large item. There is then, ample justification for an analysis of the methods and the equipment in use, for it is quite as essential that the water supply equipment shall be adequate as that the truck supply shall be adequate. Probably from the engineering standpoint it is more important, for, while production is somewhat affected by current practices, quality probably is more affected.

Basically, the water trouble on almost all concrete jobs to-day is inadequate water delivery facilities. With full production, a batch is produced every one and one-fourth minutes. Few jobs, it is true, maintain this rate consistently over any extended period, but a good many reach it not infrequently for a few minutes at a time. This rate, then, sets the water delivery rate. At least one-fourth minute is required in discharging the measuring tank. A full minute is, therefore, all that can be allowed for the delivery of the water for a batch of concrete. A 1-inch slump normally requires between 5 and 6 gallons of water per bag of cement—roughly 30 gallons per standard batch which, under the assumptions made above, is equivalent to 30 gallons a minute. In addition to this, water is required for curing. With full production and a 10-day curing period it will be necessary continually to provide for curing nearly 2 miles or about 20,000 square yards of paving. Rates of production as high as this are rare, but rates of production which will require the proper care of at least 15,000 square yards of paving though not the rule are not uncommon. To cure concrete properly requires that the earth covering shall be kept moist—not merely that it shall be wet down once in a while; and rates of evaporation from loose soil, such as that used for covering the pavement during the curing period, are high. In fact, the evaporation from such soil when the temperature is high, the humidity low, and a brisk breeze is blowing—typical summer conditions over the greater part of the United States—will often exceed one-third of an inch per day and may even exceed a half inch, equivalent to from 2 to 3 gallons per square yard of pavement. These are, of course, the highest rates of evaporation likely to be met but as it is at just these times that full protection of the pavement is most required, they must be considered the governing rates.

Finally, contractors properly prefer to conduct all their operations simultaneously. For that reason, though night sprinkling may at times be practiced, the contractor generally desires to attend to it during the day as the work is then under the observation of his regular foremen. The inspector also has a better grasp on the situation if it is done at that time, and, of course, during very dry weather a night wetting is not sufficient to keep the pavement moist throughout the following day. It is, therefore, proper to view the pavement-curing problem as one which under full production may involve the daily delivery of from 40,000 to 60,000 gallons of water in addition to mixer requirements and even under normal current production standards the daily delivery of from 30,000 to 40,000 gallons of water in excess of mixer requirements.

As shown above, full production at the mixer requires for mixing the concrete a somewhat variable amount not ordinarily exceeding 15,000 gallons per day. For current high rates of production at the mixer the requirement of the mixer seldom exceeds 12,000 gallons a day (10 hours). To these amounts there must be added a somewhat variable amount for wetting down the subgrade. It is therefore apparent that while a pump capable of delivering 100 gallons per minute has sufficient capacity to meet all needs that will ordinarily arise until the production of 18-foot pavement exceeds 750 feet a day, it is apt, at times, to prove inadequate where more than this amount of pavement is being produced unless it is worked a few hours overtime.

One thing, however, must be emphasized—namely, the adequacy of the pumping plant depends absolutely on the use of a pipe line of such size that the pressure head will always be within the limits for which the pump is designed. There are vital differences between a pump designed to deliver 100 gallons a minute against a pressure head of 100 pounds and one designed to deliver at the same rate against a pressure head of 400 pounds. This raises the question as to what pressure head is required in delivering water. Table 3 gives the pressure head in pounds for various rates of delivery in gallons per minute, per mile of common iron pipe such as is generally used on paving jobs and while the pipe is in fair condition. As the pipe becomes somewhat battered, twisted, and rusty with use, the pressure head required to force water through it will increase so that, in a general way, the figures given are minimum figures, which may be exceeded somewhat after the pipe has been in use a few years.

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[2] See Public Roads, vol. 6, No. 9, Nov., 1925, p. 198, Table 3.

Table 3.—*Pressure required at pump for each mile of pipe line to secure desired discharge, through 2, 2½, and 3 inch ordinary iron pipe*[1]

Diameter of pipe	Number of gallons to be pumped per minute, or per 10-hour day													
	20	25	30	35	40	45	50	60	70	80	90	100	110	120
	12,000	15,000	18,000	21,000	24,000	27,000	30,000	36,000	42,000	48,000	54,000	60,000	66,000	72,000
	Pressure required at pump, pounds per square inch													
2 inches	42	62	88	117	151	188	227	318	421	543	673	820	982	1,145
2½ inches	14	21	30	39	50	62	76	106	142	181	224	275	330	385
3 inches	6	9	12	16	21	26	32	44	59	75	93	114	137	161

[1] Based on Williams and Hazen's hydraulic tables.

Note.—To obtain the total pressure against which the pump must work, multiply the figures given in the table beneath the desired gallons per minute and opposite the desired diameter of pipe by the total length of the pipe line in miles and then add to this product the figures obtained by multiplying the use of the pipe line in feet from the pump to the discharge end by 0.434.

The pressures given in the table are the number of pounds, as read on the gauge at the pump, which be required to deliver the stated number of gallons per minute at the outflow end of a pipe line 1 mile long. Thus, if 50 gallons per minute are required and a 2-inch pipe a mile long is to be used, the gauge at the pump must be run up to 227 pounds. If 100 gallons are desired 1 mile from the pump and a 2-inch line is used, the gauge pressure at the pump must be run up to 820 pounds, and if this rate of delivery were required 3 miles from the pump the gauge pressure would have to be run up to 2,400 pounds. Of course. neither ordinary pipe nor ordinary pumps will stand any such pressures. The point, however, is that here is the explanation of the poor water supply so common on paving jobs to-day. Contractors are attempting the impossible. A fully adequate water supply for maximum requirements simply can not be driven through a 2-inch pipe 2 or 3 miles long. If the pump were strong enough to do it, the pipe would not stand it. The remedy lies in a larger pipe. One hundred gallons of water per minute can be driven through nearly 2 miles of 3-inch pipe with the same gauge pressure—i. e., the same effort on the part of the pump—which is required to drive it one-quarter mile through a 2-inch pipe.

With no other consideration than the water supply for the mixer, contractors could well afford to use 3-inch pipe, for its use would result in a water delivery at a pressure far less destructive of pumps, line, hose, mixer valves, etc. The added cost of pumping enough for adequate curing and, subgrade sprinkling would be so slight that the problem would cease to be troublesome, and there would probably be much less demand for permission to use chemicals as curing agents. Three-inch pipe costs about twice as much as 2-inch pipe. The change to the larger size is not, therefore, prohibitively expensive and, as suggested above, delays due to a poor water supply would be of such reduced importance as probably to much more than offset the extra outlay.

In addition to reducing lost time at the mixer, the installation of the 3-inch pipe would permit the use of relief valves which would protect the pipe line, hose, and mixer valves against excessive pressure. Gauges should also be installed at the pump, and instead of permitting pumps to run wide open their speed should be controlled according to the rate of delivery desired and the distance this delivery must be pumped. Thus, as an illustration, if the maximum water requirement is 60 gallons per minute through 2 miles of 3-inch pipe line, the pumping engine should be cut down to a speed which will produce a pressure of about 150 pounds at this delivery. The relief valve at the pump should then be set at slightly above this pressure, while the relief valve at the mixer might be set at 50 pounds or even less. This practice saves wear and tear on the pump and tends to avoid the break downs which occur most frequently when machines operate close to or above rated capacity.

The conclusion is that contractors should abandon the 2-inch pipe line and that engineers drawing specifications for paving work, if water is to be used in curing the pavement and wetting the subgrade, should give the same attention to insuring adequate pipe lines that is given to the equipment question in other phases of the work.

SUBGRADE TRIMMING DELAYS EASILY AVOIDED

The operations incident to the final fine finishing of the subgrade are performed after the mixer is moved and before concrete is placed. At this time the parting strip is placed, if one is used, and the reinforcing laid. Transverse joints are also placed at this time. These operations may take considerable time and so they sometimes hold up the mixer. A very simple expedient will avoid such delays. The boom on the mixer is about 20 feet long. Where these operations take up more time than can be had while a single batch is being mixed, it is good practice to move after every fourth batch. Four 5-bag batches will lay about 9 feet of pavement of standard 18-foot, thickened-edge section. Each move should, therefore, cover a distance of about 9 feet, and the move should leave the end of the boom as near the end of the last strip as possible. By proceeding in this manner, there always will be when the move is completed a strip of proper length (about 9 feet) at the end of the boom on which all finishing operations are completed, steel laid, parting strip and transverse joint in place, and a 10 to 12 foot space in which to work on the preparation of these items preparatory to the next move. This method, under full production, allows five minutes for final subgrading, laying reinforcing steel, parting strip, and setting transverse joint, which is a sufficient lenght of time if the men are well trained and the inspection is reasonable.

Ordinarily there is little trouble about the reinforcing steel or the final fine finish of the subgrade, but the parting strip is the source of a good deal of trouble. The cause may be traced generally to the fact that the inspector wants the parting strip set on an absolutely straight line, whereas the specifications commonly permit the use of a parting strip which is so thin that this becomes practically impossible of attainment. A more rational treatment of this matter, particularly where a finished center joint is used, would be a somewhat heavier parting strip, set slightly below the surface, capped with a fairly heavy removable crown strip, the joints of the latter being so spaced as not to fall on the parting strip joints. If, then, a good gauge is used in setting the parting strip, straight alignment can be secured.

Neither the organization of the finishing operations nor the equipment are the source of much, if any, reduction in output. They can, however, be conducted more efficiently. Such losses as result from materials pushed over the forms or otherwise spilled or wasted can be eliminated. They result from poor dumping and do not occur where there is a first-class mixer operator and a good operator on the finishing machine.

When the concrete is deposited on the subgrade it may be merely dumped in a pile, or it may be roughly spread, or, if the mixer operator really knows his business, it may be placed so nearly in correct position that it needs almost no attention from the puddlers. In case concrete of proper consistency is merely dumped, four puddlers will be worked almost to exhaustion to get it into position for the finishing machine; if a fair effort is made to spread the concrete, three men can do the work of puddling rather more easily than four will do it if no effort is made to spread it. If the mixer operator is a real expert he can place the concrete so well that two men will find it easy to handle the work of puddling. One could, in fact, do all the work if it were not for the necessity of running back and forth across the pavement in spading forms, looking after the requirements of the finisher, etc.

This not only serves to illustrate in another way the value of a good mixer operator, but what is of more importance, the value of complete and detailed training of operatives with a view to making the correct performance of one operation facilitate the correct performance of another. If the concrete is properly placed by the mixer operator, the labor of preparing it for the finisher is reduced. If it is uniformly mixed to the proper consistency, the work required in producing a proper surface is reduced. More illustration would serve only to add emphasis to the fact, outstandingly true of concrete paving work, that the total labor requirement is visibly affected by the study given to the perfection of methods, as well as to how the performance of one operation affects the labor requirements of another. Whether the men are trained to perform the various operations correctly or left to work out their own ideas as to how each specific task should be performed also affects the amount of labor required. Where no attention is given to these matters, it may still be possible to get high production—even full production—but this will be attended by the use of a good deal of extra labor. In any effort to secure high efficiency, it is necessary to go beyond the mere question of production and ascertain whether this production is arrived at through carefully studied processes which reduce the work to be done to a minimum or merely through the use of excessive labor and equipment.

THE FINISHING METHOD

Back of the puddlers, the concrete is put into shape by a finishing machine. These are of two general types—the tamping finishers and the sliding finishers. Both do good work, but as the sliding machines are substantially built, work quietly, and, on the whole, require less effort on the part of the puddlers they have proved very popular. Moreover, if the consistency of the concrete is correct and uniform these machines will produce a surface so accurate that one man with a long-handled float can do all the necessary finishing and check the work with a 10-foot straightedge, calling on one of the other laborers to help when the belt is to be pulled. This is the simplest finish which has been observed, the cheapest, and so fast that it need never lag a half hour behind the mixer. Moreover, it not only produces a very smooth pavement but does so without that long-continued working of the surface incident to some systems of finishing, which is at least in part responsible for scaling.

Here again there is an illustration of the correlation of effort necessary in order to produce a high-class product expeditiously. The success of this method and the economy it makes possible depend on two things—a proper consistency (about 1-inch slump) and uniformity in consistency. Without these the method fails. Incidentally, it may be remarked that contractors often object to a dry mix on the ground that it is hard to handle. This is not necessarily the case, but often seems to be so because a better mixer operator is required to handle the dry batch properly, and if it is not properly handled a large amount of extra work for the puddlers does result. The consistency is therefore criticized though in fact the mixer operator rather than the mix is at fault. What the contractor quite generally overlooks is that even if some extra work is thrown onto the puddlers, this is more than made up in faster and easier finishing, for where a wet concrete is used the finishing is slow and it becomes a difficult matter to secure a smooth surface. There is also trouble when some batches are wet and some dry. The wet batches settle a little while the dry batches (1-inch slump) do not settle appreciably after they are finished. The result is that direct settlement and the tendency of the wet concrete to flow after it is placed make it necessary to correct certain irregularities in the surface finish which do not appear at all when a concrete of proper and uniform consistency is used. This is outstandingly true on grades. If the consistency is right the tendency to flow ceases to be a problem.

FIG. 1.—Material removed in cleaning out mixer. Some of the blades were not visible when cleaning out was started

The above deal with some of the more important aspects of the work of the various component parts of a paving organization. Before turning to a discussion of what constitutes a proper personnel and equipment for a paving organization—matters that will be discussed in the next article—comment may be made on a few observed conditions as they affect concrete paving work. The first of these is that manufacturers would do well to inform contractors more fully concerning the spare parts which ought to be kept on hand. Ordinarily, a high percentage of all repairs involve a relatively small number of parts. On some types of mixers the cables break often. On cranes the cables and the sheaves usually give trouble. These are merely illustrative. Every manufacturer has records which show, or can be made to show, just what parts are most likely to cause trouble, and he can, if he will, tell the contractor who buys his machine which parts he ought to carry on hand as insurance against delay in case of a breakdown.

There is no machine on the market to-day so consistently designed that it will wear out without breakdowns, even if operated under ideal conditions, to say nothing of the conditions under which its work will actually be performed. Therefore, in the interest of reducing those delays due to breakdowns which are known by manufacturers to be of relatively common occurrence, they could greatly assist contractors by preparing a list of the spare parts which, in the contractor's own interest, should be on the job at all times. It should be remembered that on work of this kind the saving of an hour's mixer time, which would otherwise be lost waiting for repair parts, will ordinarily pay interest charges on a $500 investment in spare parts.

But the contractor can select these parts intelligently only after years of experience. The manufacturer is always a better judge of what parts should be on hand. His records form a sound basis for his judgment. He should, therefore, look to this matter as carefully as to the original sale. Nor should he feel that an admission that repair parts are likely to be necessary will injure sales. Every contractor knows that equipment will break down and the assistance to be had from a really well-selected list of spare parts would much more probably be accepted as a valuable service.

Another matter of general application is that the quality of the concrete is affected by the efficiency with which it is mixed. Omitting from consideration here the relative efficiency of the various types of mixers, the fact remains that a good deal of concrete of reduced quality is being produced through the use of mixers in which the blades have become badly worn or which have accumulated so much concrete on the inside of the drum that the mixing is poorly done. Figure 1 shows the material removed from such a mixer. In this particular case some of the blades were completely buried when the clean out was started.

In addition to accumulations in the mixer, accumulations all over the outside of the mixer are the rule rather than the exception. The contractor, who rides to work every morning in a well-kept car, allows his mixer, worth two or three times as much, to go uncleaned for weeks at a time. This is regretable, for, though neither the mixer nor any of the other equipment is as fine looking as the car, its proper care is quite as essential and the longer life that may be expected to result from such care is likely to more than repay its cost.

BETTER SUPERINTENDENCE THE PRIMARY NEED

Another matter that should be noted is that in all of these studies the objective has been a determination of what causes interfere with securing a full output while the mixer is working. There is also a series of causes affecting the number of hours worked that is not treated in this series of articles. It is commonly presumed that the effect of these causes of lost time is unavoidable, but the experience which the bureau's representatives have had in applying the data which has been secured in these studies in improving production has seemed to indicate that while many of the delays resulting from these causes are unavoidable some of them are not. This matter will be left for further discussion when more data are obtained.

Finally, the discussion of the causes affecting production would be incomplete without at least a brief mention of superintendence. Any construction operation divides itself naturally into at least three fields. There is first what may be termed the labor field—the work which must be done and its immediate direction. This field is occupied by common labor and skilled labor of various kinds and is directed by the various foremen. The next generally recognized field involves the business side of the work. This covers getting the work, handling the financing, and looking after the purchase of materials, securing labor, etc., in short, whatever business transactions are involved. The third field covers planning the work in its various aspects, studying methods, keeping the various operations in balance; in short, all matters that make for a more efficient conversion of raw materials into the finished product. This is superintendence. The difficulty in securing competent superintendence seems to be that the position really requires a combination of engineering training, vigorous, aggressive ability to handle men, intimate knowledge of the practical side of construction work, and courage to put into effect new ideas when it is known that these are right. That there is real profit to be had from employing an able superintendent may be seen from the fact that on a job where a superintendent meeting these requirements was employed the production of the single 5-bag paver in use averaged 95 feet of 18-foot slab an hour during the nearly 23 days (of 10 hours) it was possible to work during the month of July and that during a week's study in August production averaged a little over 100 feet an hour.

While the delays that have been discussed in this series of articles are the immediate cause of low production, the conclusion is unavoidable that most of them would long since have been greatly reduced if more attention had been given by contractors to securing really competent superintendents. As a matter of fact, superintendency is really an engineering field and young engineers who possess the proper qualifications would do well to consider the opportunities that this field offers. Contractors also might do well to consider more seriously the advisability of securing, even at some temporary financial sacrifice to themselves, men who are technically and mentally fitted to meet adequately the real responsiblities of superintendency. For at present it does not appear probable that most road construction work will be raised to that degree of efficiency which is reasonably possible until better men are more generally employed as superintendents.

PROTECTION OF CONCRETE AGAINST ALKALI

CONTINUATION OF TESTS BY BUREAU OF PUBLIC ROADS ON TREATMENT OF CONCRETE WITH TARS AND PARAFFIN

Reported by E. C. E. LORD, Petrographer, United States Bureau of Public Roads

IN AN earlier publication on the protective effect of tar and paraffin on concrete, the results of tests covering a period of approximately one year were recorded.[1] These investigations have been continued during the succeeding year and a summary of the results thus far obtained is given in the present report.

The method of treatment has been described in detail in the earlier publication and is, therefore, only briefly reviewed in the following:

Portland cement concrete cylinders (4 by 6 inches) were made up to normal consistency with Potomac River sand and gravel and graded in the proportions 1:1½:3, 1:2:4 and 1:3:6 and cured for 28 days in damp sand and about four weeks in dry air. The cylinders were all treated in batches of four each, some receiving 6 and 10 coats of water-gas tar alone while one batch from each mix was painted with 10 coats of water-gas tar followed by a seal coat of heavy coal tar cut back to working consistency with solvent naptha. The paraffin treatment consisted in immersing the test specimens of each mix for 24 hours in a 20 per cent paraffin-kerosene solution at about 80° F. and applying 4 coats of the same solution after the cylinders had dried out for one week. In one instance (1:2:4) the paint coats were omitted.

Two batches from each mix were left untreated as checks, one to be stored in alkali solution and the other in tap water.

The physical properties of the tars employed are given in Table 1.

TABLE 1.—*Properties of tars*

	Water-gas tar	Coal tar
General characteristics	Very thin liquid	Sticky semisolid.
Specific gravity (25° C.)	1.053	1.258.
Specific viscosity	3.17 at 25° C	
Float test (50° C.)		694 sec.
Softening point (° C.)		45.
Total bitumen soluble in CS_2	96.99 per cent	74.22 per cent.
Free carbon	0.30 per cent	25.62 per cent.
Inorganic matter (insoluble)	0.05 per cent	0.16 per cent.
Water	2.66 per cent	
Distillation, total to 300° C	42.20 per cent	19.0 per cent.
Residuum	57.80 per cent (semi-solid).	81.0 per cent (solid).
Specific gravity of distillate (25° C.)	0.956	

EXPOSURE TESTS

All samples were weighed before and after treatment, and stored, four each, in porcelain-lined, covered cans containing 6,000 cubic centimeters of a 3 per cent sodium-magesium sulphate solution. This solution was frequently renewed and the test specimens removed at weekly, monthly, and trimonthly periods and allowed to dry out for 48 hours before returning to the alkali bath.

At the end of the two years all samples were removed from the solution, weighed, and allowed to dry out for 10 days in the laboratory when approximately constant weight was obtained. The difference between this weight and that immediately after removal from the bath indicated capillary or absorbed moisture, while the original weight before immersion deducted from that after immersion and drying represented the quantity of secondary salts formed during the test period. These secondary salts together with the lime dissolved from the test specimens during the period of immersion, as well as insoluble residues found in the cans, served as a measure of alkali attack.

During the early stages of the investigations it was noted that the lime dissolved from the untreated test specimens gradually decreased in quantity until after a period of about three months it was entirely removed from solution. This loss in lime was accompanied by the loss of an equivalent amount of magnesia and by the formation of carbonates of lime and magnesia, and finally of calcic sulphoaluminate. It may be a matter of interest to state that the latter salt was found only as a precipitate in the cans and not in the concrete itself, thus indicating that the failure of concrete under the present test conditions must be attributed to other agencies. In the present instance the destructive compounds have been found to be magnesium hydrate and gypsum which are formed immediately when free lime, developed in the cement on setting, is brought in contact with solutions containing magnesium sulphate. This was found to be especially noticeable in imperfectly seasoned concrete, where the colloidal magnesium hydrate and gypsum crystals forming in the body of the concrete caused within three months sufficient volume change to rupture the specimens. In well-seasoned concrete, on the other hand, the formation of calcium carbonate during the curing period retarded the entrance of dissolved salts thereby reducing the potency of alkali attack.

In this connection it may be of interest to refer briefly to the chemical reactions taking place under the present test conditions. Free lime developed in the cement on setting combines with magnesium sulphate to form gypsum and magnesium hydrate. The latter salt takes up CO_2 from the solution and is converted to magnesium carbonate and finally to bicarbonate, which passes into solution and reacts with gypsum to form magnesium sulphate and calcium bicarbonate. The latter salt is unstable in solutions containing magnesium carbonate and is precipitated as normal carbonate of lime or calcite.

At the end of the two-year period the untreated test specimens were found to be incrustated with carbonate of lime and magnesia, but showed no marked indication of failure through alkali attack. For this reason it was considered advisable to postpone physical tests on all samples until decided evidence of failure occurred.

RESULTS OF TESTS

The tests thus far carried out are of a chemical nature and serve to indicate, through gain in weight by accumulation of secondary salts and absorbed moisture and loss of soluble material, the extent of alkali attack, as well as the degree of protection offered by the various treatments.

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[1] Protection of Concrete Against Alkali, PUBLIC ROADS, vol. 5, No. 3, May, 1924.

TABLE 2.—*Action of alkali solution on concrete treated with tar and paraffin after two years' exposure*

Batch No.	Mix	Treatment	Weight after treatment	Tar absorbed	Paraffin absorbed	Weight after 2 years in solution	Moisture absorbed	Secondary salt	Tota gain	Total loss
			Grams	*Per cent*	*Per cent*	*Grams*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	1:1½:3	Untreated	10, 683	----------	----------	11, 105	0. 77	3. 18	3. 95	0. 16
2	1:1½:3	6 coats water-gas tar	10, 641	0. 49	----------	10, 859	. 58	1. 48	2. 06	. 08
3	1:1½:3	10 coats water-gas tar	11, 129	. 82	----------	11, 322	. 47	1. 27	1. 74	. 09
4	1:1½:3	10 coats water-gas tar; 1 coat coal tar	11, 326	1. 12	----------	11, 429	. 27	. 64	. 91	. 02
5	1:1½:3	24 hours in paraffin solution	11, 463	----------	1. 84	11, 601	. 34	. 86	1. 20	. 07
6	1:1½:3	24 hours in paraffin; 4 coats of paraffin	11, 411	----------	2. 32	11, 524	. 32	. 68	1. 00	. 03
7	1:2:4	Untreated	10, 687	----------	----------	11, 185	. 66	4. 06	4. 66	. 19
8	1:2:4	6 coats water-gas tar	10, 872	. 46	----------	11, 178	. 31	2. 50	2. 81	. 17
[1] 9	1:2:4	10 coats water-gas tar	8, 293	1. 29	----------	8, 428	. 30	1. 66	1. 96	. 07
10	1:2:4	10 coats water-gas tar; 1 coat coal tar	11, 053	1. 46	----------	11, 178	. 23	. 90	1. 13	. 04
11	1:2:4	24 hours in paraffin; 4 coats of paraffin	11, 598	----------	2. 62	11, 724	. 26	. 83	1. 09	. 03
12	1:3:6	Untreated	10, 410	----------	----------	10, 967	. 15	4. 20	5. 35	. 26
13	1:3:6	6 coats water-gas tar	10, 505	. 64	----------	10, 847	. 63	2. 63	3. 26	. 15
14	1:3:6	10 coats water-gas tar	10, 852	2. 26	----------	11, 147	. 75	1. 97	2. 72	. 11
15	1:3:6	10 coats water-gas tar; 1 coat coal tar	10, 887	2. 94	----------	11, 045	. 43	1. 03	1. 46	. 07
16	1:3:6	24 hours in paraffin; 4 coats of paraffin	11, 415	----------	2. 77	11. 600	. 58	1. 13	1. 71	. 03

[1] Batch contained 3 cylinders.

In Table 2 are given in condensed form the results of exposure tests after a period of two years, together with the quantities of tar and paraffin absorbed, determined on a per-cent-by-weight basis. It will be noticed that the amount of protectives taken up increases with leanness of the mix and might serve as a measure of density for concrete.

Considering the quantitative effect of alkali attack it will be noted that the total percentage of gain and loss, indicated in the last two columns of the table, increases with the leanness of the mix and decreases with the quantity of protectives applied.

An interesting point is the very small percentage of dissolved material as compared to the large increase of secondary salts, chiefly carbonates of lime and magnesia and hydrated alumino silicates of lime, which exceed 4 per cent in the untreated 1:3:6 specimens (No. 12).

The amount of absorbed moisture is quite low in all samples and varies from about one-fourth to one-sixth that of secondary salts in the untreated specimens.

Regarding the relative protection offered by the various treatments, as indicated by percentage gain and loss, it will be noticed that samples of all mixes receiving 10 coats of water gas tar followed by a seal coat of coal tar (Nos. 4, 10, and 15), and those immersed 24 hours in the paraffin-kerosene solution and painted with four coats of this solution (Nos. 6, 11, and 16) show about equal resistance to alkali attack.

It will be observed also that where the surface coats have been omitted the indicated protection has been reduced nearly one-half in the case of tar (Nos. 3, 9, and 14) and about one-sixth in that of paraffin (No. 5).

As stated in a previous report, the effect of alkali attack on samples immersed in paraffin appeared to be mainly surficial, which is further substantiated by the relatively high total loss (0.07 per cent) and low percentage of absorbed moisture indicated in No. 5.

Comparing the total gain and loss of material from the untreated samples of all mixes (Nos. 1, 7, and 12) with that from samples receiving the most liberal tar and paraffin treatment (Nos. 4, 6, 10, 11, 15, and 16) the degree of protection offered by the latter would indicate at least a fourfold prolongation of the life of concrete subjected to alkali attack under conditions similar to those described.

EFFECT OF SIZE AND SHAPE OF TEST SPECIMEN ON COMPRESSIVE STRENGTH OF CONCRETE

Data of interest to those engaged in the testing of concrete will be found in Bulletin 16 of the Structural Materials Research Laboratory, Lewis Institute, Chicago, "Effect of Size and Shape of Test Specimen on Compressive Strength of Concrete," by Harrison F. Gonnerman. Tests were made on 1,755 concrete specimens at ages of 7 days to 1 year in a study of the compressive strength of—

(1) Cylinders 1½ to 10 inches in diameter, 2 diameters long.
(2) Cylinders 12 inches long, 3 to 10 inches in diameter.
(3) Cylinders 6 inches in diameter, 3 to 24 inches long.
(4) Cubes, 6 and 8 inches.
(5) Prisms, 6 by 12 and 8 by 16 inches.

The relative strength of the different forms of specimen was compared with the strength of 6 by 12 inch cylinders from the same concrete.

The principal conclusions are:

1. The 6 by 12 inch cylinder generally used for compression tests of concrete, as recommended by the American Society for Testing Materials, is a satisfactory form of specimen. However, this size of cylinder should be limited to aggregates not larger than 2 inches in diameter.

Four by 8 inch or 5 by 10 inch cylinders are suitable for the smaller sizes of aggregate. The ratio of diameter of cylinder to maximum size of aggregate should be not less than about 3. For aggregates larger than 2 inches, 8 by 16 inch cylinders or larger should be used.

2. For cylinders of length equal to 2 diameters, lower strengths were generally obtained with the larger cylinders. The decrease in strength with size of cylinder was not important for diameters of 6 inches or less; 8 by 16 inch and 10 by 20 inch cylinders gave 96 and 92 per cent of the strength of 6 by 12 inch cylinders.

3. Concrete cylinders having a ratio of length to diameter of from 0.5 to 4 gave the following average strength ratios at 28 days:

Ratio of length to diameter	0. 5	1. 0	1. 25	1. 5	2. 0	3. 0	4. 0
Strength-ratio, percentage of strength of 6 by 12 inch cylinder	178	115	107	103	100	95	90

These strength ratios agree with those reported by other investigators. The difference in strengths of cylinders having ratios of length to diameter between 1.5 and 2.5 was not important.

4. The 6 and 8 inch cubes tested at ages of 7 days to 1 year gave strengths averaging 18 and 13 per cent higher than 6 by 12 inch cylinders. The strengths for 6 by 12 and 8 by 16 inch prisms were lower at all ages than that for 6 by 12 inch cylinders; the strength ratios averaged 93 and 91 per cent, respectively.

5. For all forms of specimens the compressive strength increased with age for moist curing. For cylinders and prisms of length equal to twice the diameter or width, the 7-day, 3-month and 1-year strengths averaged 52, 142, and 178 per cent of the 28-day strength for 1: 5 concrete. The corresponding percentages for 6 and 8 inch cubes were 60, 129, and 165.

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Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.

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No. 1259. Standard Specifications for Steel Highway Bridges, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULAR

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331. Standard Specifications for Corrugated Metal Pipe Culverts.

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161. Rules and Regulations of the Secretary of Agriculture for Carrying out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D-2.	Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D-4.	Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D-6.	A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 7, D-13.	Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15.	Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

DECEMBER 31,1925

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970.097.71	$ 2.863.197.86	611.8	$ 6,885,939.73	$ 3.269.191.85	388.3	$ 8,704,688.62	$ 4,237.217.26	431.0	$ 953.521.93	$ 431.936.34	28.2	$ 3,547.911.69	Alabama
Arizona	9,580,133.43	5,016,119.94	613.8	938.773.23	573.458.56	80.4	1.276,105.71	834.777.12	92.7	176,977.10	108,150.70	26.7	3.084.742.68	Arizona
Arkansas	13,310.190.08	5,380,181.73	1,048.9	3,064.835.14	1,318.794.44	130.0	6,325,715.91	3,034,237.17	423.8	740.437.29	337,838.89	52.4	1,534,751.77	Arkansas
California	22,346.175.99	10.719.249.61	894.8	2,036.363.75	911.402.63	47.4	12.008,360.60	5.900,537.89	364.7	488,875.22	293,325.11	15.6	4.248,299.76	California
Colorado	11,876,703.94	6.067.814.34	651.2	918.425.72	492.407.79	37.7	4.450,923.10	2.226,006.88	238.0	453.284.06	254.212.34	23.8	3.285.370.65	Colorado
Connecticut	4,558.639.29	1.819.368.66	101.6	750,965.91	250,212.14	13.4	1.801,818.16	590.259.68	29.1	238,929.61	62,777.95	3.7	1,611,062.57	Connecticut
Delaware	4,281,659.81	1,496.190.65	107.1	456,744.13	212.544.95	12.3	880,042.66	388.784.75	22.7	445.559.65	9.000.00	10.5	367.537.65	Delaware
Florida	2,959,273.72	1.405.487.97	96.3				10,810,470.04	4.976.998.29	289.3	863,763.25	301.557.00	20.1	1.400.910.74	Florida
Georgia	20,156.002.37	9.406,366.46	1.478.3	2.661,700.61	1,290,934.25	157.0	11.066,415.53	5,284,563.06	699.0	1,168.539.91	519.002.21	86.5	1,931,087.02	Georgia
Idaho	9.394.676.80	4,815,332.26	600.1	211,619.32	121.958,48	24.6	3,192,163.48	2,003,022.53	206.0	626.051.34	347.904.45	17.0	1,271,409.28	Idaho
Illinois	40,010.481.10	18,640,076.28	1.236.2	933.508.56	447,871.39	32.1	8.385,506.20	4,070,001.59	284.9				6,674.248.74	Illinois
Indiana	13,639.172.65	6,562,455.68	422.1	1.132.976.57	544,542.84	41.2	17.995,239.73	8,445.342.09	492.9	543,575.62	269,346.52	16.6	2,382,667.87	Indiana
Iowa	27.272.285.21	11.107,492.99	1,996.9	722.462.52	320.365.06	36.5	7,616.193.14	3.424,369.59	471.5	1.264.775.81	604.703.90	139.5	4,028,631.46	Iowa
Kansas	26.399.695.77	9,755,273.32	831.4	3,426.659.54	1.511,580.70	206.1	10,389,493.27	4.407.022.64	491.8	3,107,841.01	716,697.19	165.5	3.073.831.15	Kansas
Kentucky	14.832.324.28	6,205,994.59	584.8	2.307.684.84	825,345.04	68.4	7,570,945.74	3,418,811.07	302.1	352,618.16	176,309.07	16.8	2,486,349.23	Kentucky
Louisiana	11,939.424.97	5,279.870.86	927.6	875,723.58	411.312.04	65.0	3.216.745.85	1.546,809.17	144.0	356.130.73	178,065.36	2.8	1.856.350.57	Louisiana
Maine	8,174.281.31	3,807.870.33	281.4	249.291.85	123,602.05	8.8	2.046,840.72	825.486.82	66.2	188.780.85	94.390.42	8.4	1,513,478.38	Maine
Maryland	8,132,506.80	3,849,383.15	294.4	396,647.75	191.101.96	15.7	2,783,283.46	1,229,741.15	125.3				654,830.74	Maryland
Massachusetts	14,047.656.22	5.467.661.28	300.6	751,829.45	271.111.11	20.4	6,104,839.26	1.591.752.67	80.4	323.228.34	105.000.00	5.3	2,673.200.94	Massachusetts
Michigan	16.234,000.80	7,328.316.91	612.6	6.532,163.80	3,001.329.49	251.8	11,328.019.35	5,200,570.40	300.8	501.034.00	208,769.57	11.9	4,603.378.63	Michigan
Minnesota	30.416.685.89	12,738.642.04	2,721.2	5.271,152.99	2,284,974.62	397.0	6,805,245.59	2,356,400.00	522.3	1,401,252.64	99,900.00	14.7	2,111,863.44	Minnesota
Mississippi	10.292.285.79	4,988,702.73	803.4	1,654,221.80	826,149.18	130.0	8,224,065.64	4.105.777.15	433.2	1,182,747.62	508,930.85	62.6	1.698,458.08	Mississippi
Missouri	17.368.156.57	8.219.411.43	1.118.8	3,849,489.25	1.863,218.92	173.4	25.608.460.96	10,054,952.66	716.5	1,134.406.92	48,617.89	29.2	2.600.235.10	Missouri
Montana	10.156.600.41	5.317,623.15	921.6	656.240.07	500,897.10	62.2	1,793,440.55	1,314,400.11	182.0	1,038,259.42	577,318.37	121.1	5,714,746.27	Montana
Nebraska	9.306,374.36	4.389,523.50	1.570.6	1,521,442.87	736,793.25	120.3	10.699,742.91	5.302,274.70	1.071.0	532.413.63	264,802.55	49.3	3,941,841.00	Nebraska
Nevada	1.917.465.69	3.088.299.78	357.3	1,162.534.50	974.820.36	95.6	4,403,970.68	3.682,501.37	398.0				1,049,593.49	Nevada
New Hampshire	4.165.687.86	1,986.226.87	208.1	568.463.14	264.530.20	20.2	853.156.53	402.397.54	26.9	9.979.13	4,989.56	0.5	511,347.83	New Hampshire
New Jersey	11,961.357.45	3.820.679.99	219.1	1.844,539.28	495.438.46	29.4	8.744,958.07	3.165,620.72	62.1				985.680.83	New Jersey
New Mexico	8,717,999.18	4.914,070.61	1.081.3	3.303,452.65	2,171.539.76	301.7	1,669,256.11	1.089.644.56	144.2	74.555.37	46,757.90	5.4	2,750,373.17	New Mexico
New York	28,697.769.67	12.229,076.53	831.6	6,679,700.73	2,743,080.06	168.8	32,742,525.75	9,904,624.05	627.9	9,110,570.00	2,230,190.00	149.1	6,938,224.36	New York
North Carolina	21.014,450.41	8.746.454.59	1,119.8	2,583,206.80	1,061,531.74	67.1	8,484.390.26	3.596.828.11	202.5	1,790,503.65	597,253.57	35.0	1,715,137.99	North Carolina
North Dakota	10.829.263.82	5,268.930.47	1,817.5	706.567.83	353,098.37	157.3	4,386.750.51	2,195,294.70	540.2	850.366.72	425.183.26	106.8	2,506.152.20	North Dakota
Ohio	41,572,252.81	15,244.993.93	1,191.1	2,821,213.23	881,893.17	85.3	11,357,815.47	4,330,272.20	345.0	1,916,946.96	661,810.54	53.6	4,502,826.16	Ohio
Oklahoma	20,787,024.84	9,672.890.34	852.2	2,022,866.21	928,040.47	87.2	7,088,346.64	3,337,040.73	318.1	719,754.38	227.746.25	20.1	1,894.068.61	Oklahoma
Oregon	14,388.188.70	7,142,364.63	794.6	1,409,879.93	767,336.36	75.3	2,817,306.00	1,644.702.27	144.2	287,542.20	5,000.00	8.8	1,319.943.74	Oregon
Pennsylvania	43,054,835.18	16,222.023.97	850.3	4,442.508.99	1,318,739.45	80.9	35,763,416.09	9,714,045.47	661.6	1,347,522.21	383,823.01	25.7	3,699,149.10	Pennsylvania
Rhode Island	2,628,496.20	1,119,688.09	64.8	481,300.30	154,988.63	6.0	2,380,016.40	654.003.63	40.2	263.012.75	63.135.00	4.2	675.753.65	Rhode Island
South Carolina	11,163,347.84	5,121,267.54	1,235.8	1,161,737.54	412,561.25	58.6	6,019,404.94	2.829,823.87	345.2	1,882,341.79	544.985.88	76.1	892,885.46	South Carolina
South Dakota	12,091,434.67	5,989.879.00	1,447.9	1,949.197.66	845,729.32	246.7	6,266,539.22	2,948,508.81	895.6				1,282,672.87	South Dakota
Tennessee	13.789,140.98	6,732.079.77	497.9	2,871,554.63	1,401,638.03	106.1	10,130.183.89	4,519,007.47	326.6	508,337.51	253,268.94	31.3	2,374,596.79	Tennessee
Texas	64,120,970.83	21,057,940.12	3,807.1	7,582,894.39	3,068,017.71	462.8	21,181,770.26	9,377,493.42	1,216.5	4,648.172.12	2,123,338.91	254.6	4,979,640.84	Texas
Utah	6,259,169.41	3,818,836.91	423.1	132,178.99	99.880.35	5.9	3,246.305.87	2,209,280.96	256.5	247,206.06	188,770.60	15.9	1,502,010.18	Utah
Vermont	3.016,174.61	1,452.894.46	107.8	200,295.47	100,082.03	3.6	1.981,542.29	884,137.49	46.2	59,192.10	29,596.06	0.1	801.796.98	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	5,698,146.03	2,668.237.33	212.7	6.158,415.87	2,703,907.73	208.1	1,080,542.11	375,834.90	32.9	1,481.535.84	Virginia
Washington	13.352,504.18	6,117,211.87	526.7	2,925,268.86	1,288,161.81	120.6	2,450,114.56	952,400.00	43.1	757,215.96	261,000.00	1.2	1,527,002.22	Washington
West Virginia	7,343.200.86	3,230,293.33	326.7	252,664.30	123,013.89	12.8	6,979,051.35	2.787,834.00	185.6	777,918.20	298.915.54	28.7	912,454.24	West Virginia
Wisconsin	21,807,140.91	8,919.640.62	1,451.7	848,974.42	419,006.01	45.7	5,813.783.15	2,859,492.42	280.8	194.866.35	97,041.00	15.3	6.143,634.95	Wisconsin
Wyoming	8,809,819.33	4,739,096.67	982.0	1,471,587.36	893,333.81	97.2	3,019,093.54	1,890,684.01	214.4	28,580.20	18,348.00		1,024,811.51	Wyoming
Hawaii							342.277.22	97,440.00	6.5				1,002,713.00	Hawaii
TOTALS	740,140,790.82	325.654,346.00	41,898.3	101,326,796.31	46,046,798.42	5,078.3	385,665,266.84	164,547,107.97	16,017.2	44,638,129.78	15,355,546.19	1,794.3	119,771,201.42	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 121,452,493.52 Federal aid $ 53.262.865.48 Miles 5,039.2

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 6, NO. 12 ▽ FEBRUARY, 1926

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

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A GENERAL FORMULA FOR WATERWAYS[1]

Reported by C. S. JARVIS, Associate Highway Engineer, U. S. Bureau of Public Roads

IN ENDEAVORING to design economical and safe bridges and similar structures the adequacy of which depends upon their proper adjustment to the flood flows of streams, the engineer is confronted with a maze of factors influencing the problem—scores of formulas and tables of data, hundreds of examples that serve as warnings, and thousands that appear satisfactory. Where records concerning the stream flow are lacking, he must rely on a study of the watershed and channel, the alluvial bars and embankments, the drift lodged along canyon walls and in crevices, and the indefinite memories of observers. In general, he will be unable to reconcile the data thus gathered; and then remains the question of assigning proper weights to various fragments of evidence.

Following such deductions, an important western railroad company located nearly 100 miles of line along a valley floor, safely above the reach of assumed maximum floods, only to experience its destruction within a few years. It was rebuilt on a higher line that would be safe against three or four times the maximum flow originally assumed. Similarly, various highway structures on Federal-aid projects regarded as entirely adequate have proved able to carry only a small part of torrential floods that have visited them.

In the course of his work in connection with the design of structures on Federal-aid road projects the writer has been impressed by the radical revisions required in waterways proportioned in accordance with the best available data and formulas. Repeatedly he has seen the capacity of channels believed to be more than ample exceeded by the floods that have followed severe summer storms; and he has been especially impressed by the variety of the indications of the various well-known waterway formulas and the narrow scope of their adaptability. In the hope of deriving a new expression capable of expansion to cover the entire range of watershed areas and conditions, he determined to plat on a single chart the best obtainable records of flood flows in relation to the area drained to ascertain the relation of the various common expressions to the data thus charted, and, if these were found to be lacking in agreement, to develop, if possible, a new and more adequate expression. At the start he had no preference for any one formula.

OVER A THOUSAND FLOOD RATES CHARTED

More than a thousand maximum flood rates were selected from the great mass of available data, fully three times that number, to represent the entire possible range of drainage areas and volumes of discharge. These observations were platted with q, the yield in second-feet per square mile, as ordinates and M, the drainage area in square miles, as abscissas, and on the same chart the graphical curves of the standard run-off formulas were superimposed so as to bring out the relation of each expression to the field of use. As originally reported in a paper entitled "Flood Flow Characteristics," published in the Proceedings of the American Society of Civil Engineers, December, 1924, this process brought forth a modification of the Myers formula with a coefficient varied according to a decimal scale as the simplest and most reliable and the only one capable of convenient extension over the entire scope of practice.

The original Myers formula, published in 1879, was the first American waterway expression. It employed the square root of the drainage area in acres and applied to this factor as a multiplier a coefficient ranging from 1 to 4 according to the surface condition to represent the area of waterway required in square feet. If a be chosen to represent the area of waterway in square feet, A the drainage area in acres, and C the coefficient depending upon the surface condition, the original form of the formula may be rendered as,

$$a = C\sqrt{A}$$

If the drainage area, A, in acres be converted into square miles, M, and the velocity of water through the waterway, V, be assumed at 10 feet per second, the discharge in cubic feet, Q, may be represented as—

$$Q = aV = 10C\sqrt{A} = 10 \times 25.3C\sqrt{\frac{A}{640}} = 10 \times 25.3C\sqrt{M}$$
$$= 250C\sqrt{M} \text{ (approx.)}$$

The claims regarding the wide application of the Myers formula brought forth the criticism of the late A. M. Wellington, who had no faith in such a device. His counterproposal was to make an intelligent estimate of the waterway required and then prescribe double this area for drainage structures. But, apparently, the "intelligent estimate" must depend upon some scale of comparison, whether it be tabular data, formulas, or field observations and experience; and while none of these can take the place of mature judgment, they are regarded as guides and controls, as later recognized by Mr. Wellington.

Since the development of the original Myers formula several others have been devised upon the basis of conditions observed in particular locations, among them those of Fanning, Talbot, Kuichling, Murphy, McMath, and a number of others.

Reducing the better known of these expressions to the same form—the form to which the original Myers formula has been reduced above—they may be expressed as follows:

Myers: $Q = 250\ C\sqrt{M}$
Fanning: $Q = 200\ \sqrt[6]{M^5}$
Talbot: $Q = 127\ C\sqrt[4]{M^3}$
McMath: $Q = 174\ C\sqrt[5]{M^4}$

The values of q, as derived from these and other formulas using the ordinary values of C, are plotted in Figure 1 against the corresponding values of M. Superimposed upon these are selected maximum flood flow data representative of the entire mass of data collected.

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[1] See also the paper by the writer entitled "Flood Flow Characteristics," Proc. Am. Soc. Civ. Engrs., December, 1924; and discussions in subsequent issues of the same publication to December, 1925.

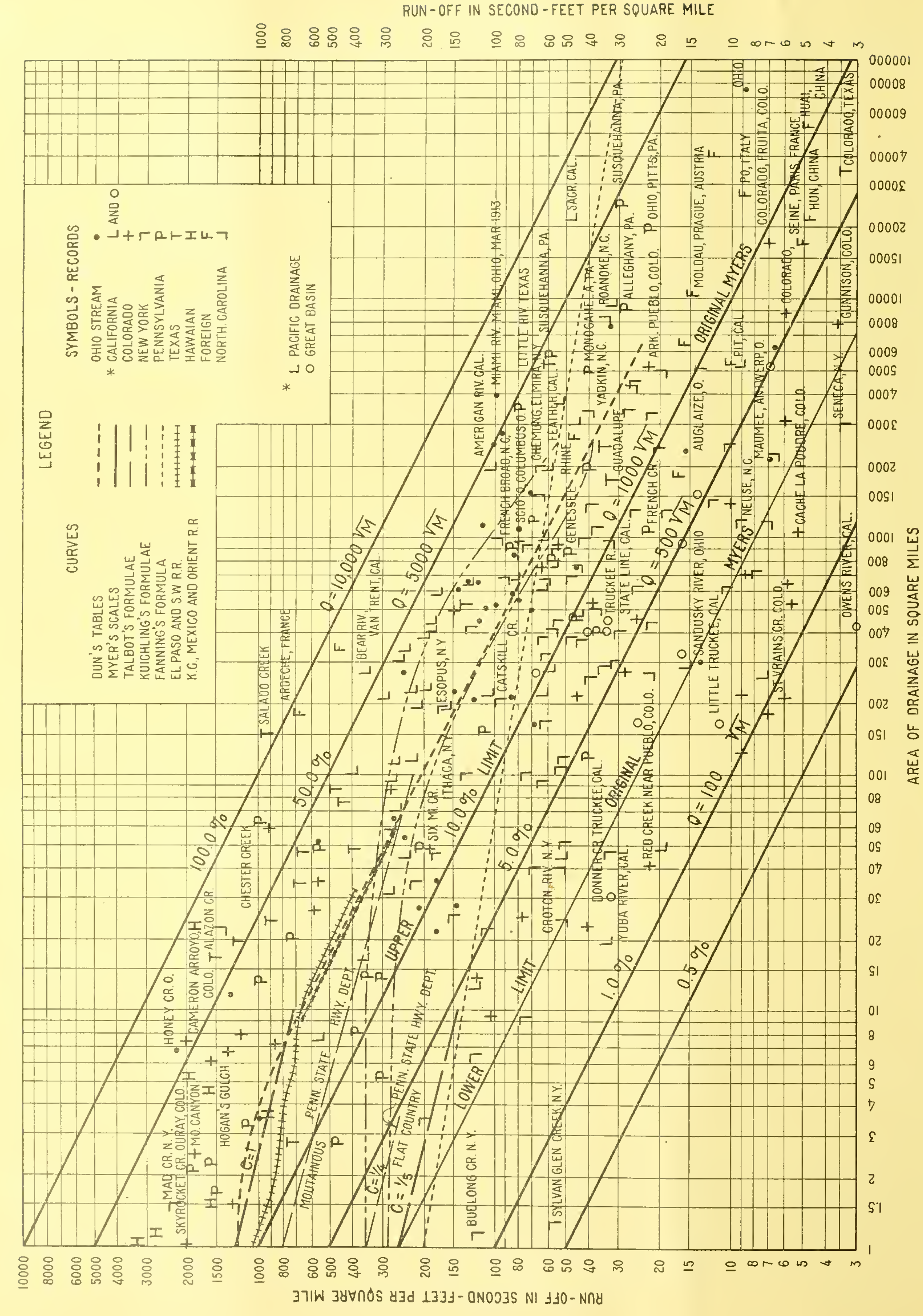

Fig. 1.—Graphic representation of selected run-off formulas, including the original and modified Myers formulas, and maximum run-off of representative streams

It will be observed from Figure 1 that the principal weakness of the most popular expressions for run-off is their limited scope, both geographically and numerically. It is typical of the best known formulas that C has no variation or a range of 4 or 6 at most, as in the original Myers and Talbot's, and it is apparent that such a range is not great enough to cover the field of the maximum flood flow data.

It will be observed, however, that the values of q as derived from the original Myers formula do bear a relation to the drainage area throughout the range from 1 to 100,000 square miles which more closely conforms to the mass of plotted data than the corresponding relation as determined by the other formulas, although it also is deficient in that the range of values of C is not great enough to include all the plotted points.

MODIFICATION OF THE MYERS FORMULA

Recognizing this fact it has been felt that a modification of the original Myers formula by increasing the range of the values of C might convert it into a form in which it would be suitable for adoption as a general run-off formula. A value of C equal to 40 being tentatively assumed as a maximum the resulting graph is found to lie above practically all the plotted points representing flood run-off rates and to conform closely to the enveloping curve, and the assumption of 40 as a maximum value for C is in this way justified. Substituting this value of C in the typical formula the resulting equation becomes:

$$Q = 250 \times 40 \sqrt{M} = 10{,}000 \sqrt{M}$$

Similarly for the lower limit, the locus of the point $q = \frac{100\sqrt{M}}{M}$, which corresponds to a value of C of 0.4, conforms well with the general slope of the band of platted run-off points, and lies practically at the lower limits of the band. These observed relations of the two assumed values of C to the field of the platted data suggest the idea that the modified Myers formula might be considered as

$$Q = R\sqrt{M}$$

in which R is the expected rate of run-off in second-feet from 1 square mile, varying between 100 and 10,000.

Moreover, the fact that the ratio of the maximum and minimum assumed values of C is as 100 to 1, and that these values just include the entire field of the platted run-off data, suggests the further possibility of comparing stream discharges according to the percentage, p, of the maximum for the corresponding area as obtained from the modified maximum formula, $Q = 10{,}000\sqrt{M}$, which percentage is readily scaled in Figure 1 for any point platted. Adopting this idea, the final form of the revised formula becomes

$$Q = 10{,}000\ p\sqrt{M}$$

and p becomes a percentage coefficient the value of which is to be determined for each particular watershed. As a variant form the equation may also be cast as $Q = 100\ P\sqrt{M}$, in which P is the numerical value of the percentage rating.

This general formula is apparently applicable to all areas from 1 to 100,000 square miles or more. For areas less than 1 square mile it should be converted into the form, $Q = 100\ P\ M$, otherwise discordant results will be obtained. This flattening of the slope of curves for small areas is characteristic of Murphy's, Kuichling's, Cramer's, Possenti's, and in fact all except the straight-line formulas. Dun's tabular values, also, show when platted that the yield is nearly proportional to the drainage area for all areas up to 3 square miles, but varies as the square root, approximately, for all greater values of M up to 6,500 square miles. (See fig. 1.)

Thus the modified Myers formula is mainly a transformation to the percentage basis and an extension of the original to cover the entire scope of ordinary practice. There may be other formulas just as capable of extension and modification to cover the entire field, but it will be difficult to find one more readily applicable, and agreeing with the platted run-offs more consistently. That such an extension of the scope is necessary is obvious from Figure 1. As well might we speak of a single unit strength for all materials of construction as to claim that a single value of a coefficient in any formula is applicable to the entire field of design. This fact was evidently recognized by Kuichling, Talbot, Dun, and Myers, for they each devised a range of values for C, thus defining a strip or zone as shown in Figure 1; but these zones are shown by the data recently collected to be much too narrow. Thus, the original Myers expression had a range of only from 2½ to 10 per cent as measured on the new percentage scale, and the other well-known formulas are similarly limited.

The superiority of the modified Myers scale in this respect and the necessity of a range as great as that suggested are illustrated by the wide range of the Ohio data platted in Figure 1. It is apparent that the Myers scale covers the field of design in Ohio more thoroughly and satisfactorily than any of the other formulas represented, yet the Ohio State highway bridge specifications approved June 10, 1925, for Federal-aid projects, prescribe the use of Talbot's, Kuichling's, and Fanning's expressions, none of which is adequate to cover all the Ohio run-offs represented in Figure 1.

MYERS FORMULA APPLICABLE TO WIDE RANGE OF AREAS

The most important claim made for the Myers formula is that it appears to express more accurately than the others the effect of the area of the watershed upon the rate of run-off. The variable coefficient C expresses the effect of various conditions of the drainage area with respect to vegetal cover, soil porosity, slope, storage, etc., but assuming these conditions to be approximately constant a single value of C may be used to determine the run-off from areas of widely different size.

From an exhaustive study of available hydrographic data there is considerable evidence of close similarity between the Myers scale ratings applicable to various sections of the same watershed or river valley and even between different rivers in the same general territory. Thus the Allegheny, Monongahela, and Ohio Rivers and many of their tributaries seem to have attained nearly 30 per cent, and marked departures from this rating are readily accounted for by abnormal conditions. Both the St. Lawrence and the Niagara, on the other hand, rate nearly 6 per cent as the result of storage in the Great Lakes; but it is a

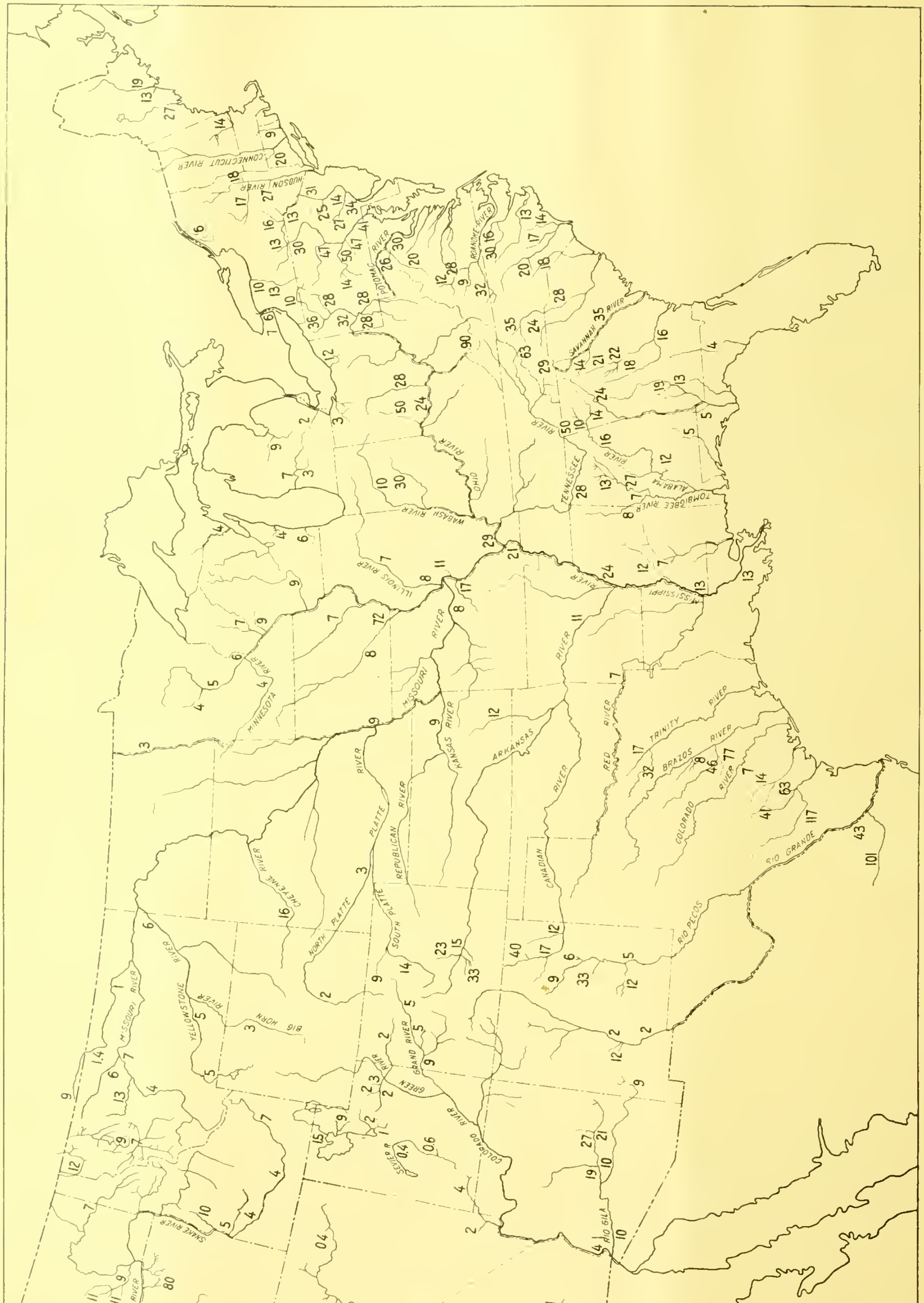

Fig. 2.—Myers scale ratings corresponding to maximum observed flood flows of streams in the United States

significant fact that their tributaries also range from 2 to 14 per cent generally, and average about the same intensity as the main rivers. The surface run-off is thus seen to be not more than a fourth of that which prevails in the Ohio drainage basin; yet within the two systems the ratings are relatively constant.

From numerous observations such as the above it appears that the Myers ratings as established for any given section of a drainage basin may often be applied to a more or less remote section of the same basin or even to a section of another but similar basin with considerable confidence in the approximate accuracy of the discharge as computed for areas of widely different magnitude. Percentages derived in this way must, of course, be regarded as first approximations only, subject to corrections representing changes in climatic, topographic or other physical characteristics which either detain or augment flood crests.

As a practical example of the possibilities in the use of the modified formula in this way there is the case of the Springerville bridge over the Little Colorado River in Arizona. A permanent masonry bridge had been designed which provided a waterway twice as great as that of the old structure. From all readily available data this would accommodate the highest stage of 50 years; and the designer therefore felt justified in adopting high embankments for approaches.

The representative of the Bureau of Public Roads who checked the proposed structure discovered that the waterway capacity would rate less than 3 per cent on the Myers scale, whereas several streams originating in the same mountains and somewhat similar with respect to the character of their watersheds, but of unequal drainage area, had been observed at from 10 to 21 per cent on the Myers scale. This prompted further investigation and it was established that the old bridge had accommodated only a minor portion of the maximum flood, for the adjoining meadows had functioned as a spillway too deep for fording. As a result the new bridge was given a larger waterway, and in addition one high approach embankment was eliminated and a by-pass substituted with capacity equal to that of the main structure. Thus at slight expense the flood channel was more than doubled, and the danger of impounding against a high earthen embankment and of sudden failure was removed.

Another instance that may be cited is that of the Rockville crossing of the Susquehanna a few miles north of Harrisburg. This crossing presented a complex problem, not only because of the high flood yields from this part of Pennsylvania, but because of the prolific and conflicting data according to the usual reckoning. It is well known that for small areas in this district Talbot's formula had to be expanded, by the use of a coefficient, C, equal to 2 or more in order to express the run-off rate; but that gave no direct information regarding the comparable flood volume from 24,030 square miles.

Expert analysis finally resulted in the adoption of a design providing a waterway corresponding to a value of C equal to 0.25 in Talbot's formula, or one-eighth of that which obtains for small areas. Records from distant stations could not contribute; they were unsatisfactory even as checks upon the computations for the bridge site in question, because they involved such dissimilar coefficients; therefore an independent solution was required.

ADAPTABILITY OF THE MYERS SCALE

Contrast that method with the one based on the Myers scale. According to this standard the ratings for typical areas on the headwaters, and also for such a major tributary as the Juniata at Newport seem to approach 50 per cent as a limit, and thus the presumption is that at least this intensity should be the basis of design for a permanent structure. Confirmation of this conclusion is affored by the high record observed at Harrisburg in 1889, which attained 47.4 per cent.

It is important to note that a value of C equal to 0.25 in Talbot's formula corresponds to 40 per cent on the Myers scale for 24,030 square miles, so that the adopted basis of design is not far different from the one above derived. Furthermore, due weight should be accorded the influence of rapids at this crossing, thus warranting a higher velocity than 10 feet per second. A 50 per cent rating of waterway on the Myers scale with V equal to 10 is equivalent to a 40 per cent rating with V equal to 12.5 feet per second.

With a view to their usefulness in this way the Myers scale ratings corresponding to maximum observed flood flows of a large number of streams in the United States are presented in Figure 2. To anyone who appreciates the elements of detention and concentration as affecting run-off, the prevailing 9 per cent shown for the Missouri River is as appropriate as 50 per cent for the Miami, 24 to 32 per cent for the Ohio, and 0.4 per cent for the Humboldt River under desert conditions. Also, the ratings of 5 per cent around the Mississippi headwaters among glacial débris, 11 per cent at St. Louis, and 13 per cent in Louisiana are in line with what should be expected at the present stage of river control and land reclamation. The decrease of lateral storage by reason of new dyke inclosures will doubtless add to the severity of flood stages in the lower valley, and invite disaster if the Ohio River and other main tributaries reach maximum stages at the same time. This may not have occurred within the past century, but nevertheless it is properly included in the reckoning.

An abrupt change of percentage caused by the entrance of a tributary subject to violent floods may display the adaptability of the Myers scale as definitely as does the uniform rating of other streams throughout their courses. Thus, the Mississippi below Cairo rates 21 per cent, a compromise between the 11 per cent and 29 per cent attained by the main stream and tributary above the junction; but the Scioto, on the other hand, registers a uniform maximum of 26 per cent between the limits of 50 and 1,000 square miles of drainage area.

IDENTICAL PROBLEMS SOLVED BY VARIOUS FORMULAS

To illustrate the divergence of standard run-off expressions, and to test them all on the same basis, five identical problems representative of widely varying conditions have been solved and the results are recorded in Table 1. In general, it will be observed that each expression with the exception of the Myers formula serves satisfactorily for a restricted class of drainage areas, but fails to give consistent solutions for widely different conditions. The analyses by the Myers formula, on the contrary, yield results all of which are regarded as acceptable first approximations of the authoritative data, which were withheld from the writer until the analyses had been made.

Unpublished data collected by the division of agricultural engineering of the Bureau of Public Roads, provided the approved solutions for the Catoma, Marais des Cygnes, and Kootenai waterways; while for the Rockville crossing of the Susquehanna and the bridge over the Rio Grande at Brownsville, Tex., the high record floods observed during at least 30 years have governed. Reference to soil maps and a study of actual field conditions should furnish means for closer and more reliable estimates. The solutions recorded as obtained by the use of the Myers scale were dependent mainly upon the fragmentary information afforded by precipitation records, topographic maps, and the tabulations, illustrations, and discussions appearing in recent issues of the Proceedings of the American Society of Civil Engineers under the title of "Flood Flow Characteristics".[2]

A brief outline of the methods used would necessarily include reference to Table 2 of the above-named paper, which shows that the Verdigris River at Liberty, Kans., nearly 100 miles southerly from Ottawa, rated 9.1 per cent; the Neosho at Iola, midway between these two stations, rated 12.2 per cent; and the Kansas River at Lawrence, 30 miles northward from Ottawa, has been observed at 9.3 per cent. Comparison of topographic and precipitation data, together with fragmentary information as to surface conditions, hydrology, and vegetation, led to the conclusion that the extreme rating on the Marais des Cygnes at Ottawa would be 14.1 per cent, requiring provision for a flood flow of 49,880 second-feet. The highest flow of record was reported as 59,000 second-feet, of which nearly one-fourth was estimated discharge in the lateral channels.

METEOROLOGISTS' VIEWS OF THE MYERS SCALE

From preliminary studies it even seems probable that some relation exists between the percentage of rainfall which appears as run-off and the rating on the Myers scale. Expert meteorologists have indorsed this manner of expressing flood intensities, because it is consistent or even comparable with the factor of yield for a drainage basin.

For many watersheds the annual rainfall in inches multiplied by the average yield factor is numerically equal to the maximum rating on the Myers scale. It is recognized, of course, that this is purely an accidental relation. Thus central Florida with a mean yearly rainfall of 50 inches on porous soil, flat slopes, and dense vegetation yields about a tenth as run-off, and rates up to 5 per cent as shown in Figure 2. For southwestern Texas the corresponding figures are 15×0.15=2.25; for southern Ohio, 40×0.75=30; for northern Ohio, 35×0.20=7; for central Michigan, Wisconsin, and Minnesota, 30×0.20=6; for eastern Montana, 15×0.40=6; for north-central Nevada, 6×0.10=0.6; for the upper Sacramento Valley, Calif., 50×0.55=27.5; for northwestern Oregon, 90×0.50=45; for southeastern Pennsylvania, 45×0.60=27; for western North Carolina, 65×0.60=39; for eastern North Carolina, 48×0.25=12; and for eastern Maine, 40×0.45=18. For Pueblo, Colo., the maximum record attained during 1921 so far exceeded the 50-year mean of 11.56 inches as to merit adoption, and accordingly 20.28×0.70=14.2. All of the above would serve as first approximations to the percentages of rating on the Myer scale as recorded in Figure 2. Such methods of analysis should not, however, be allowed to take the place of thorough investigations, as special local conditions may alter the run-off intensities to a marked degree.

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[2] Issues Dec. 1924

TABLE 1.—*Functions of the drainage area involved as a factor in each run-off formula, and typical solutions for waterway area derived therefrom*

Run-off formula	Area exponent		Waterway area required for run-off from 1 acre (0.0015 square mile)	Streams included in the test				
	Exact	Approximate		Catoma Creek, Ala.	Marais des Cygnes (Osage), Ottawa, Kans.	Kootenai, Bonner's Ferry, Idaho	Susquehanna, Rockville, Pa.	Rio Grande, Brownsville, Tex.
				Area of watershed in square miles				
				285	1,247	13,000	24,030	230,000
				Area of required waterway				
			Square feet	*Square feet*	*Square feet*	*Square feet*	*Square fete*	*Square feet*
Kuichling		−1	0.05 – 0.1	3,330–5,730	6,300– 10,700	10,800– 30,000	32,300– 52,500	207,000–505,000
Lauterberg	1		0.0002– 0.01	8 – 164	290– 500	2,130– 4,000	5,000– 10,000	100,000–200,000
Peck (former Mo. Pac. R. R. practice)	1		0.17 – 0.25	30,400–45,600	133,000–199,000	1,360,000 –2,040,000	2,563,000–3,845,000	24,500,000–36,800,000
Murphy		−1	0.024	2,630	5,600	24,100	40,700	350,000
Fanning	5/6		0.23	2,280	7,482	54,600	90,000	575,000
McMath	4/5		0.1 – 0.6	2,620– 9,000	10,000– 32,500	60,000– 208,000	100,000– 336,000	700,000– 2,050,000
Talbot and Burkli-Ziegler	3/4		0.16 – 1.0	1,450– 8,600	4,600– 27,500	26,000– 156,000	40,000– 240,000	215,000– 1,290,000
Fuller		0.8	0.1 – 3.7	2,200–10,500	6,500– 30,000	40,000– 190,000	65,000– 300,000	360,000– 1,700,000
Metcalf & Eddy	0.73		0.66	2,850	9,000	52,000	89,000	460,000
Chamier, Hawkesley, and Gray	3/4							
Tidewater Ry., Virginia	7/10		0.62	4,030	12,000	66,600	106,000	514,000
Churchill-Wentworth	2/3		1.0	3,200	8,660	40,800	61,600	273,000
Cooley	2/3							
Cramer		2/3						
Dun's tables	1-1/2		0.2 – 0.3	2,820– 4,580	5,670– 9,220			
O'Connell	1/2							
Dickens	1/2		0.9	376	786	2,540	3,450	10,700
Kutter		1/2	0.6	2,370	4,950	16,000	21,700	67,200
Hering, Dredge, Possenti		1/2						
Italian formulas		1/2						
Myers (original)	1/2		1– 4	425– 1,700	890– 3,560	2,900 – 11,500	3,900– 15,600	12,000– 48,300
Modified Myers	1-1/2		0.15– 15.0	170–17,000	356– 35,600	1,150– 115,000	1,560– 156,000	4,830– 483,000
Modified Myers with appropriate values of *P*	1-1/2		0.15 – 15.0	1,710	4,988	14,160	75,000	48,000
Authoritative solutions,[1] with $V=10$				1,440	5,900	12,600	65,000	38,000

[1] Not divulged until after analyses had been made with aid of modified Myers formula.

The Lauterberg formula, published in 1887[3] attempted to relate run-off to yearly rainfall. It may be written as $Q=0.082\ ChM$ where h is the total annual rainfall in inches and C varies from 0.2 to 0.7 for marshy and mountainous country respectively. The typical solutions shown in Table 1 illustrate the futility of that particular method of approach. Most of the formulas therein listed come within hailing distance of correct results in at least one column; but the absurd errors that attend other examples have left most of them in disrepute. The flexibility of the Myers scale and the soundness of the analysis based thereon have passed the preliminary tests, and justify further consideration. A similar claim can be made for the Talbot, Kuichling, Murphy, Dun, Kutter, Wentworth, and various other run-off expressions within the special and restricted fields for which they were devised.

ADVANTAGES OF THE MYERS SCALE

By adopting the two elements usually published by the United States Geological Survey and common to all run-off expressions, or easily derived therefrom, the Myers scale provides a means of comparison—a common denominator, so to speak—for all of the complex formulas in current use. Some of these take into account the assumed rainfall intensity, the slope, soil porosity, channel roughness, lateral storage, vegetation, axial directions of drainage and storm movements, time required for flow from the most remote parts, and the shape of the watershed. Outstanding is one similarity: They all seek either the required area of waterway or else the volume of flood discharge, and therefore take account of the run-off rate and the drainage area, the two coordinates of the Myers scale. Their conflicting methods of analysis and their limited scope have resulted in confusion and perplexities for investigators. For example, the Fanning formula for floods will rate 2 per cent on the Myers scale for 1 square mile, and 50 per cent for 16,000 square miles, while Talbot's formula with C equal to 0.2 rates 2.5 per cent and 28 per cent for the same areas respectively.

A study of the data presented in this article and the paper published in the Proceedings of the American Society of Civil Engineers, previously mentioned, warrants the following conclusions with respect to the advantages of the Myers formula as modified:

1. Run-off rates in California, Colorado, New York, North Carolina, Ohio, Pennsylvania, Texas, various foreign lands, and therefore all countries have been shown to be equally well represented on the Myers scale.
2. The use of various divergent formulas for any district will produce inconsistencies by neglecting certain portions of the desired field and overlapping others, as illustrated in Table 1.
3. The long recognized need for a general run-off formula to cover all conditions of practice may be met by the Myers scale.
4. Flood ratings on the Myers scale for any river section may provide a clue to run-off intensities elsewhere on the same watershed, irrespective of area. This is the most important claim set forth.

NEED FOR FIELD INVESTIGATIONS

As pointed out by the committee of the American Society of Civil Engineers investigating the failure of the South Fork Dam[4] there are other potential Johnstown floods that may be averted by prompt and intelligent action. Since that time both State and national supervision of such structures and channels have extended with obvious benefits to all concerned; yet there remains a field for important service by experts in a general reappraisal of the flood situation, guided by recent happenings and developments in analysis, forecasting, and design.

No doubt the proponents of each successive formula have been hopeful of its adoption for practical design purposes, yet the need for a general expression has not heretofore been satisfied. The Myers scale has withstood the preliminary trials, but the critical test of time and experience that comes only with time is yet to come; and its limitations are clearly recognized by all who have followed its development and application.

There has never been a more hopeful prospect for correlating and digesting the great mass of isolated hydrographic data than now lies before us, with interest widespread on account of recent flood disasters, with the special committee on flood protection data appointed by the American Society of Civil Engineers, and the governmental agencies well equipped and organized, and only awaiting financial support. A million dollars wisely spent now for such a purpose would furnish the best insurance against flood dangers to millions of citizens and billions of property values, and would warrant a corresponding decrease in annual premiums to underwriters.

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[3] Proc. Inst. C. E. vol. 149, p. 392; quoted in 1911 Proc. Am. Ry. Engr. Assoc., vol. 12, pt. 3, p. 494.

[4] Trans. Am. Soc. Civ. Engrs., vol. XXIV, June, 1891, p. 431.

URBAN ASPECTS OF HIGHWAY FINANCE[1]

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The first section of this paper, dealing with the special interests of cities and urban motor-vehicle owners in the highway finance policies and practices of State and county governments was published in the January issue. The remaining sections dealing with the methods and problems of urban highway finance and with certain financial aspects of the traffic-congestion problem are combined in this article.

THE most formidable difficulty which research in the problems of highway finance encounters is the lack of adequate statistical data, and this lack is even greater for urban than for rural highway finance. Such data as are available are rarely sufficiently detailed or suitably classified to serve effectively the purposes of research in the problems of urban highway finance, and it is, for instance, impossible to determine with any close degree of accuracy the amounts of urban highway expenditures and the sources of urban highway revenues for the larger American cities.

URBAN HIGHWAY EXPENDITURES

Table 1 presents the statistics of highway expenditures in 1923 of all American cities over 30,000 in population, as compiled by the Federal Census Bureau. To obtain all-inclusive figures, it would be necessary to add to the total of $324,607,000 shown in the table a substantial item for interest on highway indebtedness, another substantial item for the portion of police department expenditures incurred in connection with traffic regulation, and a minor item for the costs of pavement construction and maintenance, snow removal, and street sprinkling incurred by electric railways in carrying out their franchise obligations. There are no data upon which to base even rough estimates of the amounts involved in these additional items, but it seems to the writer a reasonable guess that if these were added the total highway expenditures would not fall short of $400,000,000 per annum. If there be added the expenditures of the thousands of incorporated places under 30,000 in population, the total figure might well reach $450,000,000, or about 45 per cent of the total expenditures on rural highways.

TABLE 1.—*Highway expenditures of American cities over 30,000 in population, 1923* [1]

	Expenditures	Total
OUTLAYS		
Streets, roads, and alleys	$177,010,000	
Other highway structures	23,722,000	
All other	3,911,000	
		$204,643,000
EXPENSES		
Supervising departments	2,388,000	
Roadways	52,069,000	
Other highway structures	11,603,000	
Prevention of street dust	3,189,000	
Snow and ice removal	8,277,000	
Street lighting	34,967,000	
Waterways	902,000	
Repair and construction for compensation	6,569	
		119,964,000
		324,607,000

[1] U. S. Bureau of the Census, Financial Statistics of Cities, 1923.

CITY STREET EXPENDITURES NOT INCREASED BY MOTOR VEHICLES

Table 2 presents data illustrating the trend of per capita highway expenditures for 146 cities for which continuous comparable data were procurable. If these cities can be taken as representative of the general urban situation, per capita urban expenditures for highways have increased only moderately in the last 20 years, and if allowance is made for the decline in the purchasing power of the dollar they have decreased. Such increase as is shown in the table has been confined to the postwar period, and much of it could reasonably be explained as due to high prices and to an attempt to make up for the enforced curtailment even of urgent expenditures during the period of the war, without reference to other causes. The cities have been increasing their expenditures on other services much more rapidly than on highways. This is in sharp contrast to the situation with respect to rural highways, for there has been since the advent of the automobile a very marked increase, both absolute and in relation to State and county expenditures on other activities, in the per capita expenditures on rural highways. A rough estimate shows, for example, that the per capita expenditures of the American people on rural highways were about $2 in 1910, $5 in 1920, and $8.50 in 1923. This contrast would appear to indicate that the development of motor transportation has exercised a much less marked influence on urban than on rural highway expenditures.

TABLE 2.—*Per capita expenditures for highway purposes of 146 American cities, 1903 to 1923* [1]

Year	Outlays	Expenses	Total	Percentage of total expenditures for all purposes
1903	$3.62	$1.64	$5.26	27.7
1905	2.87	1.67	4.54	22.4
1907	3.26	1.91	5.17	22.5
1909	3.29	1.71	5.00	21.4
1911	3.79	2.04	5.83	22.9
1913	3.39	1.93	5.32	21.4
1915	3.76	2.06	5.82	21.7
1917	3.25	1.96	5.21	20.1
1919	2.41	2.04	4.45	16.3
1922	4.85	2.87	7.72	16.8
1923	5.22	3.01	8.23	17.3

[1] Computed from data in U. S. Bureau of Census, report on Financial Statistics of Cities, 1923.

The cities have been growing rapidly in population, and therefore in density of population per square mile. On the supposition that an explanation of the moderate increase in recent years in the per capita highway expenditures of cities might be found in the increasing density of urban population, Table 3 was constructed to test the hypothesis. If the increase in density of population, other things remaining the same, tends to reduce the per capita highway costs, the per capita expenditures for highways should vary, as between cities of different population, inversely to population. Table 3, however, indicates that there is no significant difference in the highway expenditures per capita between cities grouped according to size of population, and fails, therefore, to confirm the hypothesis. An examination of the detailed data for individual cities likewise fails to reveal any tendency of per capita highway expenditures to vary inversely to the size of the city.

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[1] Part of a report presented by the writer before the annual meeting of the Highway Research Board National Research Council, at Washington, D. C., Dec. 3, 1925.

TABLE 3.—*Per capita highway expenditures of cities classified according to population, 1923*

[Computed from data in U. S. Bureau of the Census, report on Financial Statistics of Cities, 1923]

Population	Outlays	Expenses	Total
500,000 and over	$4.77	$3.37	$8.14
300,000 to 500,000	6.31	3.32	9.63
100,000 to 300,000	5.19	2.80	7.99
50,000 to 100,000	5.70	2.50	8.20
30,000 to 50,000	5.51	2.80	8.31

The failure of the urban statistics to disclose any such marked influence on urban highway expenditures of the growth of motor transportation as is apparent in the statistics of rural highway expenditures is perhaps to be explained by the following factors, which are presented as tentative hypotheses and not as observed facts:

1. The development of motor transportation has increased traffic on rural highways relatively more than on city streets and has therefore made necessary relatively greater increases in expenditures on rural than on urban highways.
2. The city street systems, at least in so far as width, mileage, and substructures were concerned, and possibly also with respect to type of surface, were better prepared to meet the demands of motor transportation than were the rural highways prior to the modern era of highway improvement.
3. On city streets the adjustment to the increased volume of traffic has been made in large degree by permitting congestion to develop and by restrictive legislation, whereas on rural highways, extension of facilities was more flexible, because it was not seriously hampered by high cost of the additional land necessary for such extension nor by the location thereon of expensive buildings, and adjustment has consequently been effected in greater degree by providing increased facilities for traffic.

SOURCES OF MUNICIPAL HIGHWAY REVENUES

American municipal expenditures for highway purposes are in the main financed out of the general revenues of the cities, and even where special funds exist for highway purposes the published statistical returns often fail to segregate them. It is impossible, therefore, to ascertain even approximately for the cities as a whole the specific sources from which their highway revenues are derived, and the amounts and proportions from each source. There is available, however, some material which if analyzed indicates in a general way the sources of municipal highway revenues.

Special motor vehicle taxes.—There are no compilations of the amounts of revenue derived by American municipalities from either special municipal motor vehicle taxes or from grants or refunds from State motor vehicle taxes. The census report on Financial Statistics of Cities for 1923 shows, however, that the receipts in that year of all American cities over 30,000 in population from "general licenses" amounted to $12,417,001, and a comparison of the detailed figures given under this head with evidence from State and city financial returns indicates that revenues from motor vehicle license taxes comprise at least 90 per cent of this amount, and that the figure given includes the share of cities in the receipts from State motor vehicle license taxes as well as the revenues from the few municipal motor vehicle taxes, which are independent of, or additional to, the State taxes.

An analysis of the detailed data from this and other sources leads to the estimates that in 1923 American cities over 30,000 in population shared in the receipts from State motor vehicle license taxes to a total amount of not less than $7,000,000 and not more than $8,000,000, and that these cities received from separate municipal motor vehicle taxes not less than $4,000,000 and not more than $4,500,000. Only seven cities over 100,000 in population imposed municipal motor vehicle license taxes, namely, Chicago, St. Louis, Kansas City, Louisville, Omaha, Richmond, and Memphis, and these seven cities collected approximately $4,000,000, of which approximately three-fourths was collected by Chicago alone. To these amounts should be added shares of the cities in State gasoline taxes and also receipts from special municipal taxes on both public and private vehicles, for which no data are available. It is assumed that $10,000,000 is a generous estimate to cover these additional items for the year 1923.

Receipts from highway privileges.—The Census Bureau reports for 1923 receipts from highway privileges for all cities over 30,000 in population totaling $26,700,000. These cover payments from steam and electric railroads (also from bus and taxi companies for the privilege of using the streets) and receipts from public utilities for the privilege of placing wires, pipes, poles, and other equipment on or under the streets, charges for the privilege of maintaining vaults under sidewalks, etc.

Receipts from earnings of highway departments.—The Census Bureau reports for 1923 receipts from earnings of highway departments of all cities over 30,000 in population a total of $7,955,684, of which $7,211,235 covered receipts in compensation for repair and construction, not explained but probably referring to pavement repairs and construction required or made necessary by and compensated by electric railways and by public utilities locating their equipment in the subsurface.

Receipts from subventions and grants by other civil divisions.—The Census Bureau does not separate receipts of cities from subventions and grants by other civil divisions for highway purposes from receipts for other purposes but the total receipts in 1923 of all American cities over 30,000 in population from State and county subventions and grants for other purposes than education amounted to $10,294.276. It is probable that the great part of these grants was for highway purposes, and it will be estimated that highway grants amounted to $10,000,000.

Receipts from special assessments and special charges for outlays.—The Census Bureau does not separately classify the purposes for which special assessments are levied. In 1923 the total receipts, of all cities over 30,000 in population, from special assessments were $122,273,060, of which $117,966,561 were for capital outlays and $4,306,505 for current expenses. Some of these receipts were from assessments for sewers, parks, and other nonhighway purposes, but the predominant use of special assessments by American cities is to provide funds for highway purposes, and it is a conservative estimate that of these receipts $100,000,000 were for such purposes.

TABLE 4.—*Estimates of highway revenues of American cities over 30,000 in population by sources compared with highway expenditures, 1923*

	Revenues	Percentage of total expenditures
HIGHWAY EXPENDITURES		
Outlays	$204,643,000	
Expenses	119,964,000	
Total	324,607,000	
HIGHWAY REVENUES		
Apportionment of receipts of State motor vehicle license taxes	[1] 7,500,000	[1] 2.3
Municipal motor vehicle taxes	[1] 4,250,000	[1] 1.3
Apportionment of receipts of State gasoline taxes, and municipal taxes on bus and truck lines and vehicles for hire	[1] 5,000,000	[1] 1.5
Receipts from highway privileges	26,700,000	8.2
Receipts from earnings of highway departments	7,955,000	2.5
Receipts from State and county grants	[1] 10,000,000	[1] 3.1
Receipts from special assessments and special charges for outlays	[1] 100,000,000	[1] 30.8
Other sources	[1] 163,202,000	[1] 50.3
Total	324,607,000	100.0

[1] Estimated.

In Table 4 the estimates of highway revenues by sources are tabulated and compared with the total highway expenditures of cities. It should be remembered that the statistics of highway expenditures do not include the interest payments on highway indebtedness, the expenditures of police departments on traffic regulation, nor the costs to electric railways of the paving and other highway services which they are required to contribute. On the other hand, the total figures for highway revenues do not include the value of the highway services rendered by electric railways. The amount attributed to "other sources" must come in the main from property taxes or from receipts from bond issues, and the bond issues will in the main eventually be liquidated from the receipts of property taxation. In 1923 over 92 per cent of the tax receipts of American cities over 30,000 in population was derived from property taxes. If this percentage be applied to the figure in Table 4 for "other sources" and if it be assumed that all State and county grants to cities for highway purposes are derived ultimately from motor vehicle taxation, the estimate is reached that the highway expenditures for 1923 of American cities over 30,000 in population were, or would eventually be, financed: 45.3 per cent from taxes on property; 30.8 per cent from special assessments on property; 10.7 per cent from highway earnings; 8.2 per cent from motor vehicle tax revenues; and 5 per cent from other sources. If the estimate that the inclusion of omitted items would raise the total to $450,000,000 be accepted, the percentages would be about as follows: Property taxes, 59 per cent; special assessments, 22.2 per cent; motor vehicle taxes, 5.9 per cent; highway earnings, 7.7 per cent; other sources, 5.2 per cent.

MOTOR TRAFFIC AND URBAN HIGHWAY FINANCE

This situation contrasts sharply with the method of financing rural highways, especially because of the small percentage of urban highway revenue which comes from motor vehicle taxation as compared to the 50 per cent or so of the rural highway revenues which are derived, either in actual fact or in equivalence, from Federal and State taxes on motor vehicles and gasoline. Are there any valid reasons why motor vehicles should contribute in so much smaller proportion to the cost of city streets than to the cost of rural highways?

In the first case, the use of rural highways for other purposes than motor transportation is now negligible, whereas city streets are used to an important extent by pedestrian traffic, electric railways, and in some cities horse-drawn vehicles. If the principle of charging the costs of highways to users were rigidly followed, all of the sidewalk costs and a substantial cost of the crossings at intersections should be charged to pedestrian traffic. The city itself should also bear a part of the costs proportionate to the use of city streets by municipal fire, garbage disposal, police, and other service vehicles. The electric railways and horse-drawn traffic should also contribute.

Second, rural highways serve no other purpose than transportation whereas city streets serve a variety of other purposes. They are the means of access of light and air to the adjoining buildings. They serve as fire barriers between city blocks. Their surface and underground areas are used as the locations for the equipment of most public utilities, telegraph and telephone poles and wires, water, sewage, and drainage mains, gas mains, and electric wires, etc. Where they are parked or boulevarded, or where trees and lawns are maintained within the street area, the streets serve as elements in the beautification of the city and as recreation areas for the city population.

Third, most of the highway services such as street lighting, abatement of dust, removal of snow, street cleaning are not made necessary solely by the existence of vehicle traffic, and serve, not only such traffic but also pedestrian traffic and the occupants of adjoining buildings. Rural highway services to other than vehicular traffic are negligible, and under some cicumstances rural highways are a detriment rather than an advantage to immediately adjoining property.

ITEMS OF STREET COSTS CHARGEABLE TO MOTOR VEHICLES

It has already been shown that the per capita highway expenditures of the cities have not increased greatly since the advent of the automobile, and that if allowance be made for the decline in the purchasing power of the dollar they have actually decreased. It is undoubtedly true, however, that the per capita expenditures, such as they are, are greater than they would be if there had not been so tremendous a development of motor transportation. The principle supported by the highway finance committee of the National Tax Association that the costs of rural highways should be borne by the users thereof is applicable to urban highways in the same way and for the same reasons *to the extent that the fundamental conditions are similar*. The costs of providing urban facilities for motor transportation, to the extent that these facilities are made necessary by the growth of motor transportation and serve no other important purpose than that of facilitating such transportation, should be met by charges on the users of such transportation.[2] What proportion of the total urban highway expenditures is properly to be charged to motor vehicles it is impossible to estimate with any reasonable degree of accuracy until more detailed statistics of the objects of such expenditures are made available and until those in charge of the operation of the various urban highway services analyze these operations with a view to ascertaining the relative degrees in which various urban activities benefit therefrom.

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[2] Subject, however, to the qualification made later with respect to special assessments.

The items in the urban highway costs which can with most certainty be charged to motor vehicles are the costs of construction and maintenance of roadway pavements, of street widenings made necessary by the growth of motor traffic and of traffic regulation. It seems doubtful that motor traffic has a sufficient degree of responsibility for any of the other items in the highway bill to justify imposing upon it the cost thereof. Even for the items here specified, certain important deductions should in equity be made. Other types of transportation using the paved surfaces should share the costs with motor traffic in proportion to use thereof and damage thereto. The city should meet out of its general tax revenues a portion of these costs to cover the pedestrians' share therein. There should not be charged to motor traffic any repair or other pavement costs made necessary by operations in connection with the subsurface utilities. The city should also meet out of general or departmental funds a share of these costs proportional to the use of the roadways by its own vehicles. Motor traffic should be credited, toward its share of these costs, with whatever revenues the city may receive from State or county which are derived from motor vehicle taxation.

SPECIAL ASSESSMENTS IN URBAN HIGHWAY FINANCE

The estimate was made above that special assessments levied against land assumed to benefit from highway improvements and levied in proportion to the assumed benefits, in 1923, met half the cost of "outlays" or capital expenditures for durable highway improvements. The "benefits" to land from highway improvements are ordinarily not benefits separate and distinct from the direct benefits of the improvements to the users thereof, but are a different manifestation of the same benefits. The benefit to land from a highway improvement is for the most part merely the result of the ability of the landowner to extract from the user of the improvement all or part of the monetary value of the improvement to the latter. Land adjoining a new highway improvement rises in value precisely because it is anticipated that its owner will be able to perform such an extraction, and it rises in the measure of such anticipation. To the extent that there is a benefit to land, there is, with minor qualifications, an equivalent subtraction from the net benefit to the user of the improvement.

If a highway improvement is financed by special assessments against actual increases in land values unmistakably resulting from the improvement, this makes certain that the cost of the improvement shall be paid out of that portion of the benefit to the users of the improvement which the landowners expect to be able to appropriate for themselves, but in the absence of friction and assuming the accurate assessment of costs against benefits to adjoining land, it is theoretically possible that charges against users and assessments against benefited land, whichever method is adopted, shall in the final incidence be borne fully by the owners of the benefited land.[3]

Assuming the possibility of the satisfactory administration of special assessments, they are generally preferable to taxes on highway users, because they reach the benefits where they concentrate in relatively few hands and where the entire spread of the benefits over the duration of the improvement is at once capitalized and thus made available for immediate assessment. Where a highway improvement of a durable sort results beyond reasonable doubt in a substantial increase in the market value of neighboring land, it is clearly more equitable to charge the cost of the improvement against the benefited land, but with the amount of the benefit to the land as a maximum for the assessment, rather than to spread the cost through general property taxation on property of all kinds and locations, regardless of its share in the benefits resulting from the improvement. To most persons also it would seem more equitable to charge the costs against the benefits to land rather than against such benefits as the landowners permit to remain with the users.

SPECIAL ASSESSMENTS MORE APPROPRIATE FOR URBAN THAN RURAL HIGHWAY FINANCE

There is much greater scope in urban than in rural highway finance for the use of special assessments as a substitute for taxes on highway users to meet the costs of durable highway improvements. Special assessments can not be properly administered unless there result from the improvements which they are intended to finance substantial, concentrated, and readily ascertainable increases in the value of land in the immediate neighborhood of the improvement. This is much more likely to be the case for street improvements than for improvements to rural highways. The chief benefit to land values from a new rural highway may be in the urban centers at its extreme limits, or the benefits may be spread lightly over a wide area extending across county and even State lines. In rural highway finance the only effective way of reaching the important beneficiaries of improvements is to tax the immediate beneficiary—the highway user. In urban highway finance, special assessments may often be a more certain and more convenient method of achieving this purpose.

There is need of caution, nevertheless, in the use of special assessments. It is generally taken for granted that because ordinarily they are subject to the legal principle that the assessment must not exceed the value of the benefit and because their administration is always subject to certain legal restrictions intended to safeguard the assessee, special assessments are always in fact, what they must be in law, special charges against special benefits. There is reason to believe that in most cities where special assessments are much used, there is inadequate technique for ascertaining the existence, the location, and the amount of benefit, and the special assessment tends to become merely a special land tax levied over an arbitrarily delimited area and with erratic variations of rates as between different parcels of land. Very often what appears to be an increase in land value due to an improvement may upon examination turn out to be merely part of a general rise in land values and often a fictitious one due to the decline in the purchasing power of the monetary unit.

Very often the anticipations of landowners with respect to the stimulus which a projected improvement will give to land values transpire after the event to have been mistaken. Many improvements are competitive in their effect on land values. An improvement in locality *X* gives it an advantage over locality *Y* which shows itself in a rise in land values in X. Locality Y thereupon undertakes a similar improvement and its land values again come to a parity with values in X. But X has now lost its temporary superiority and its land values fall back to their original level. The result of an investigation covering a number of years would under such circumstances show little or no effect on land values in X and Y of the improvements made by them, but the usual technique of special assessment, which deals only with prospective benefits of an improvement not yet made and disregards the effects of the improvement on land values outside the area of supposed benefit, would here find a proper basis for the levy of benefit assessments. All persons with special-assessment experience know of instances where improvements financed by special assessments have lowered the market value of the assessed land because of the assessment burden for which it was made liable. It has been only the general rise in land values, due to growth of population and to the general rise of prices, which has kept the shortcomings of the special assessment as commonly administered from receiving the serious attention which they call for.

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[3] This assumes that the users of the improvement are "ultimate consumers" of the service it renders, for example, travelers for pleasure, and do not pass the benefit on to employers of their services or to purchasers of their products, as well as to adjoining landowners. For other than pleasure traffic, charges on users are theoretically preferable to special assessments, because they will be passed on in part at least to all the ultimate beneficiaries of the improvement and not to one class of beneficiaries, landowners, alone.

TAXATION OF ELECTRIC RAILWAYS FOR HIGHWAY USE

It is the common practice in American cities to levy a privilege tax on electric railways for the privilege of using the city streets, or to require them to construct and maintain at their own expense the paving within the track space and for a specified distance on each side. They are also required in some cities to remove the snow and to sprinkle the streets on which they operate. The item "receipts from highway privileges," amounting in 1923 to $26,700,000, includes amounts received from electric light, telephone, and water companies for the privilege of using the surface or subsurface of streets for their structures and equipment, and also receipts for the privilege of maintaining vaults under the streets; fruit, gasoline, and other stands on the streets; and awnings, signs, etc., extending over the sidewalk. But a large portion of these receipts consists of amounts paid by electric railways for the privilege of using the streets.

In 1919, after which year the Census Bureau returns ceased to differentiate between the various types of highway privileges, the revenue from charges for the use of space on or under the highways by privately owned public utilities, mainly electric railways, amounted to 93 per cent of the total receipts from highway privileges, as compared to 7 per cent from charges for the use of space for miscellaneous special purposes, such as awnings, gasoline pumps, signs, etc. It has been estimated, also, that the annual cost to American electric railways of rendering the paving services required by their franchises exceeds $20,000,000. This probably includes the item amounting to $7,211,000 for 1923 of compensation to city highway departments for repair and construction services, most of which undoubtedly came from electric railway companies which, instead of doing their own paving, had it done for them at their expense by the municipal highway departments. In a few cases, as for instance Chicago, the city also receives a share of the receipts of the surface railroads.

The electric railways make, therefore, a substantial annual contribution to the cost of urban highways. Their representatives, in fact, complain that they make too large a contribution, especially as compared to motor transportation, and that this discrimination in taxation operates as a subsidy to competing methods of urban transportation. They protest especially vigorously against the paving requirements, which they characterize as an obsolete survival from the days of horse cars, when the horses did actually wear out the pavements.

There is no evident reason why electric railways should contribute more heavily in proportion to their use of city streets than other types of transportation, and it is in fact desirable that competing types of transportation should bear the highway costs properly attributable to them in proportion to their use of the highways, in order that their relative capacity to render transportation service should be tested under equal conditions. The fact that electric railways are common carriers, whereas private automobiles are not, should have no bearing on the question, since the special privileges enjoyed by a common carrier are granted in the public interest rather than in the interest of the carrier and are accompanied by special and onerous obligations and restrictions. But if electric railways are being required to make too great a contribution to highway revenues, it is only true, if true at all, in comparison with other types of transportation. In so far as the paving and snow removal requirements are concerned, they are clearly arbitrary and have no necessary relationship to the costs to the cities resulting from the operations of electric railways. Under existing conditions, electric railways should be required to meet the highway costs incurred by the cities on their behalf to the same degree as such requirement is imposed on other types of vehicular traffic using city streets.

In so far as paving is concerned, it is proper to attribute to electric railways such increase in the costs of construction and maintenance of pavement as result from the presence of tracks and the operation thereon of street cars. How this increase can be computed is a problem for the engineers, but the type of test suggested by some engineers, namely, a comparison of the paving costs on two streets of similar width, one with and one without street-car tracks, is clearly defective unless the character and volume of vehicular traffic on both streets is constant and unless the same standard of maintenance is applied to both streets. But in large cities, and especially in the congested portions thereof, space utilization is a more important economic factor than the wear on pavements, and a thoroughgoing apportionment of highway costs would take into account the comparative utilization of space of the different types of carriers as well as their wear on the pavements.

FINANCIAL ASPECTS OF URBAN TRAFFIC CONGESTION

A careful survey of the American literature on the traffic congestion problem has made it clear to the writer that the explanation of the causes of traffic congestion and the appraisal of the comparative merits of the many proposals which have been made for its solution are primarily technical problems for the engineer and the transportation expert to deal with. There are, nevertheless, some angles, of the problem which are at the same time important and of special concern to municipal finance, and with these I propose to deal briefly.

Any program of highway improvements to remedy traffic congestion raises four fundamental financial questions:

1. How much will it cost?
2. Is there any alternative program which would bring greater relief at the same cost, or the same degree of relief at less cost?
3. Is the relief it will bring of sufficient economic importance to warrant its cost?
4. Who should pay the cost, and how?

Estimating the costs of public improvements and the results of such improvements on traffic conditions is of course a technical task which belongs presumably to the highway and traffic engineers. The question: Who should pay the costs and how, has already been dealt with at some length. It may be added, however, that whether the municipal government meets the costs in the first instance by taxation or by borrowing, the long-run costs to the community as a whole will in either case be the same. For the policy of financing an expensive program of highway improvements by borrowing it can be argued that it is not as likely as the pay-as-you-go method to lead to the costly and uneconomic postponement of the making of improvements until long after the need for them has become urgent. The voting public is almost everywehere more favorably disposed toward projects for highway or other major public improvements if they are to be financed by borrowing instead of by current taxation. On the other hand, it can be argued for the pay-as-you-go policy that it is less likely to lead to a premature or overambitious program of expenditures. Ordinarily, however, if the improvement program is extensive and can not conveniently be carried out in gradual stages over a period of some duration, it can not in practice be financed out of current taxes and must be either financed by borrowing or abandoned. The arbitrary debt limits to which many cities are subject often operate as insurmountable obstacles to the execution of urgent programs of highway improvement. There remains the most fundamental and the most difficult question of all, namely, is the project worth its cost?

WHAT IS MEANT BY TRAFFIC CONGESTION

The first requisite for an adequate analysis of the problem of traffic congestion would appear to be a careful definition of "congestion." The nearest approach to a formal definition which I have been able to find in the literature is the following: "The meaning of the term 'congestion' as applied to traffic conditions in this report is that degree of overcrowding of vehicles in streets that obstruct freedom of circulation, with attendant consequences of economic waste, and inconvenience." But maximum freedom of circulation, convenience and economy for an individual vehicle is to be obtained only if there are no other vehicles on the road. This is defining congestion by calling it overcrowding, which is not very helpful. There are two different senses in which the term is commonly used: (1) To indicate such a volume of traffic on the roads as to reduce below its potential maximum the speed at which traffic moves, and which I will call "retardation of traffic," and (2) To indicate the presence on the roads of so great a number of vehicles as to reduce the "traffic capacity" of the roads, whose consequences I will term "supppression of traffic."

The term "traffic capacity" of a street is used to indicate the number of vehicles per hour which can be passed through at a given point in the street. The traffic capacity of a street increases sharply with increases in the speed of movement of traffic until a speed of about 14 or 15 miles per hour is reached. At speeds higher than this the traffic capacity of the street steadily decreases with increases in the speed of movement because of the increase in the safe braking space between vehicles. For an uninterrupted stream of traffic, the theoretical traffic capacity at a speed of traffic movement of 14 or 15 miles per hour appears to be about 125 per cent of the capacity at 30 miles and about 140 per cent of the capacity at 40 miles. Below 15 miles per hour, speeding up of traffic therefore increases traffic capacity, above 15 miles per hour it decreases traffic capacity.[4]

Traffic congestion therefore has two phases: (1) The retardation of traffic, and (2) the suppression of traffic. An increase in the number of vehicles on the road always tends to retard the rate of movement of the traffic. If the increase in the number of vehicles goes beyond a certain point it not only retards traffic but it reduces the amount of traffic which can be passed through the street per hour. Where the only speed restriction is that which is the automatic result of the number of vehicles on the road, reduction of speed retards traffic until a minimum of speed of about 14 miles per hour is reached, but increases traffic capacity; further reduction of speed not only retards traffic but it also suppresses traffic by reducing traffic capacity.

There are to be found scattered through the literature on the traffic problem numerous estimates of the economic loss to different urban communities resulting from the prevailing traffic congestion. Recently an estimate of $2,000,000,000 per year for the United States as a whole due to traffic congestion and improper control of traffic facilities has been given wide publicity. These estimates of the costs of traffic congestion commonly ignore the "suppression of traffic" phase of traffic congestion, although they deal almost exclusively with traffic areas where congestion has retarded the rate of traffic movement to far below the rate of maximum traffic capacity and has therefore resulted in considerable suppression of traffic. Though they profess to be estimates, therefore, of economic costs of retardation of existing traffic, in no case that I have encountered is any indication given of the basic speed, 10 miles per hour, 30 miles per hour, 60 miles per hour, or whatever it may be, from which the degree of retardation, and by inference the amount of time and money lost, are measured. Acceptance of the current estimates as reasonably accurate would be much easier if it were made clear just what it is that they estimate. In any case, estimates of the costs of congestion should take into account the economic loss due to suppression of traffic, which, for all we know, may be more important than retardation of traffic. The development of a satisfactory technique for estimating the costs of traffic congestion will not come until congestion is analyzed and dealt with in quantitative rather than qualitative terms.

REMEDIES FOR TRAFFIC CONGESTION

Any program for the relief of traffic congestion should be written in terms of the volume and character of traffic for which provision is intended to be made and the standard of provision which it is planned to give to it. Provision should be made, of course, for anticipated expansion of traffic, estimated as best it can be from such factors as population trends, automobile registration trends, per capita passenger mileage movements per annum as density of urban population increases, etc. Estimates of prospective needs for traffic facilities are often so made as to imply that all that is sought for the future is the avoidance, as the volume of traffic grows, of any intensification of the existing degree of congestion. Most such estimates, moreover, overlook the stimulus to traffic which results from the extension of traffic facilities of itself, and which would lead to an increase in traffic after the extension was made even though pouplation, motor vehicle registrations, and other such basic factors remain constant.

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[4] Cf. Regional Plan of New York and Its Environs, Highway Traffic in New York and its Environs, Lewis and Goodrich, pp. 80 ff.

The methods proposed by traffic engineers for the relief of traffic congestion fall into five classes: (1) Improved traffic guidance, (2) minor improvements to existing traffic facilities, (3) major extensions of highway facilities, (4) zoning and decentralization of business, (5) traffic restriction. Any expense involved in traffic guidance and in minor improvements to existing traffic facilities, such as removal of obstructions, laying of smoother pavements, narrower sidewalks where roadways are congested and sidewalks are not, easier curves at intersections, through modifications in sidewalk corners, are clearly justified where they will bring an appreciable measure of relief.

Zoning can be used to ameliorate traffic conditions by decentralizing traffic and by reducing the need for transportation. It must, however, be gradually applied and conservatively administered if it is not to impair seriously existing real-estate values and if it is to receive the necessary degree of support from public opinion. Relief to existing traffic congestion by zoning must always, therefore, be a slow process, a matter of decades if not of generations. Its major contribution to the solution of the traffic problem must be sought in its use to forestall prospective intensification of traffic congestion by preventing further overconcentration of traffic-producing enterprises in narrowly circumscribed areas. It has an important and constructive place in the long-run program, but its potentialities are limited in dealing with the congestion which already prevails. In congested areas of large cities major improvements are liable to involve great expenditures, because more land for street space can ordinarily be acquired only at prohibitive cost and in many locations is wholly out of the question, while the cost of construction of elevated or subsurface traffic ways is many times greater than the cost of the existing natural surface facilities. In many instances, however, the only alternatives are serious traffic congestion growing progressively worse as the city grows in population, a tremendously expensive program of major highway improvements, or traffic restriction. In most American cities adjustment is being made to the pressure of expanding volume of traffic by a compromise between these three.

For most, and probably for all, large American cities a program of providing in the congested areas ample facilities for all the traffic, whatever its type, which would offer itself if the facilities were there, would involve so staggering a cost that such a program would clearly be impracticable. It is clearly uneconomical also to tolerate the persistence of a degree of traffic congestion so great as to reduce substantially the traffic capacity of the streets. The long-run program of dealing with the traffic problem must necessarily provide both for extension of facilities and for restriction of traffic. The general sentiment in support of the free and unrestricted use of the public streets is powerful, and headway against it can be made only very slowly. Nevertheless, traffic restriction is inevitable. If it is not applied by traffic officials in accordance with a carefully designed plan, it will come about automatically and in greater degree through the suppression of traffic resulting from acute congestion.

To what extent in any particular situation the problem of traffic congestion should be met by extension of facilities and to what extent by restriction of traffic should be determined only after careful study of the situation and the application of as scientific a technique as can be developed for the comparison of the costs of the improvements with the economic costs of traffic restrictions if the improvements are not made. It is an unfortunate element in the situation that in most American cities the imagination of the public is more easily captured by projects for expensive ornamental driveways and boulevards in outlying sections of the city than by the more prosaic but usually much more urgent improvements which would serve to give substantial relief at the points at which traffic congestion is most acute. There are few American cities in whose congested areas an immediate and extensive program of major highway improvements planned to furnish an increase of traffic facilities is not economically justifiable. As land values rise fairly steadily in the traffic centers of large cities and as the process of replacing old and moderately sized buildings by new, more expensive, and higher buildings progresses, the cost of major improvements requiring the utilization of increased land space becomes greater, and the need for such improvements becomes more intense. In such cases delay is usually very expensive.

THE NEED FOR TRAFFIC RESTRICTION

The common practice, nevertheless, of measuring the extent of traffic facilities needed by the amount of traffic which would be present if the facilities were there is dangerous, because it fosters the delusion that traffic facilities are costless or that provision must be made, regardless of the cost, for all the facilities which traffic may demand. There is a scope for traffic restriction as one of the means of meeting the problem of traffic congestion. On purely economic grounds traffic restriction is always clearly preferable to the suppression and acute retardation of traffic which result from extreme traffic congestion. Up to a certain point, which differs with circumstances and can be determined only approximately and only by careful and expert survey of the situation, traffic restriction is more economical than the extension at great cost of existing traffic facilities. Traffic restriction would suppress traffic, but properly applied it would differ from the suppression of traffic resulting from acute congestion because it would not be accompanied by an impairment of the traffic capacity of the existing highway facilities, and because it would select the traffic to be suppressed in accordance with the economic importance of different types of traffic instead of arbitrarily.

In congested areas, what most needs economizing is not wear of the pavement but space utilization. The primary object of traffic restriction should be so to control the volume of traffic as to maximize the traffic capacity of the congested highways. There should be no restrictions on any highway, therefore, unless the volume of traffic in the absence of restrictions is so great as to retard the speed of traffic movement substantially below the rate at which traffic capacity would be at its maximum for that highway. Where maximum traffic capacity can be maintained only by the application of traffic restrictions, the restrictions should be applied to various types of traffic and of carriers in inverse order to their utilization of road space per unit of transportation service rendered. In congested areas speed above the rate which brings maximum traffic capacity is to be regarded as an expensive luxury and to be given little extra consideration; inability to maintain that optimum speed is on the other hand an expensive nuisance and should be penalized. Where congestion is exceptionally acute, comfort and convenience of passengers must also become a minor consideration and must yield to movement of traffic if there is conflict between them. As traffic conditions ordinarily vary widely as between different periods of the day the restrictions also should be made to vary according to traffic conditions, being intensified at the traffic peaks and lightened or wholly removed at the traffic troughs.

Many estimates have been made of the relative efficiency in terms of space utilization per unit of transportation service rendered, of the different types of passenger carriers, but in no case that I have seen have these estimates taken into account all the factors which require consideration in estimating space utilization, or been based on tests made under conditions which permit of decisive determination of the most economic use in terms of passenger transportation to which a given stretch of highway can be put. What they do show conclusively enough is that in congested areas pedestrian traffic makes economical use of space per person per mile traveled, and that parking cars and loading and unloading of freight carriers at the curb and on the sidewalk is the most extravagant form of space utilization. They show also beyond reasonable doubt that private automobiles and taxis, with their average load in every city under two persons and a large fraction of that load consisting of chauffeurs, are much less economical users of congested road space per passenger mile than either motor busses or electric street cars. As between electric surface railways and motor busses, however, the evidence which is commonly brought to bear in favor of one or the other is contradictory and inadequate for conclusive determination of their relative economy as users of road space.

THE TEST OF UNECONOMICAL SPACE UTILIZATION

The proper test of relative economy in use of space has not yet been definitely worked out, and to some extent it must probably be a different test under different circumstances. The most common test applied to different types of passenger carriers using surface way is square feet of space occupied per seat. This is inadequate in a number of respects. Among the additional factors which should always be given consideration are: The possibilities of reasonable overload at traffic peaks; the speed per mile in conjunction with the corresponding minima of side clearance and safe braking distance; and the interference with other types of traffic. If vehicles are not permitted on congested highways if their width plus their necessary clearance exceeds the maximum width available on such highways per lane of traffic, width is a negligible factor and lineal feet tests are more conclusive than square feet tests.

The data presented in Table 5 indicate how different types of passenger carriers meet some of these tests of economy in space utilization.

TABLE 5.—*Street space utilization of various types of passenger carriers* [1]

[Data from Daniel L. Turner, consulting engineer, report to New York Trans Commission, May 9, 1923, and from other sources]

	Square feet per seat	Square feet per passenger	Lineal feet per seat	Lineal feet per passenger
Standard street car	6.63	[1] 3.32	0.79	[1] 0.39
Double-deck bus	3.28	[2] 2.19	.41	[2] .27
Ford touring car, 2 passengers			5.83	
Packard touring car, 2 passengers			8.33	
Pedestrian		1.96		1.25

[1] 100 per cent overload. [2] 50 per cent overload.

These data take no account of necessary clearances and stopping spaces, potential speed in heavy traffic, and interference with other traffic. They assume that the motor bus is capable of a 50 per cent overload above rated seating capacity, which is perhaps open to question. The double-deck bus with an uncovered upper deck in bad weather can not attain even its rated seating capacity, but the development of a satisfactory covered upper deck would remove this handicap. Test counts made by the Chicago Surface Railways Co., of the upper-deck passengers on Chicago motor busses during heavy rain showed in the count that 16 busses with partially-covered upper deck averaged 26.1 passengers on the upper deck as compared to an average of only 4.4 passengers on the upper deck of 48 uncovered busses at the same time on the same routes. Another count made during rain, snow, and sleet showed 10.9 passengers on the average on the upper decks of 22 partially covered busses as compared to an average of 2.8 passengers on the upper decks of 64 uncovered busses.[5]

In Chicago the average speed of busses in the Loop district was shown by tests to be 5.81 miles per hour, as compared to 6.21 miles per hour for surface cars, or about a 7 per cent superiority for the electric cars. Outside of the Loop district the busses averaged 11.87 miles per hour as compared to 11.63 miles per hour for street cars,[6] but in Chicago the busses outside the Loop operate on routes more favorable to speed than those of the street-car system, namely, parks and boulevards with rights of way, and with few intersecting streets. The evidence as to necessary clearance and stopping space and interference with other traffic is too contradictory and based too much on inadequate tests to have much weight one way or the other. The most conclusive test, if it were practicable, would be to test the space utilization unit of transportation service rendered, of the different types of carriers by putting through a selected mile of highway, during successive hours and under similar conditions with respect to traffic guidance, cross traffic at intersections, etc., its maximum capacity of:

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[5] Computed from manuscript report of tests lent to the writer by the Chicago Surface Railways.

[6] Data from Report of John A. Beeler, consulting engineer, to New York Transit Commission, January, 1923.

1. Motor busses alone.
2. Private passenger cars alone; and combinations in varying proportions, of—
   - (*a*) Motor busses and electric street cars.
   - (*b*) Motor busses and passenger cars.
   - (*c*) Electric street cars and passenger automobiles.
   - (*d*) Electric street cars, passenger cars, and motor busses.

In each case each vehicle should be loaded, or be presumed to be loaded, with its potential maximum load at rush periods. Such a test would disclose conclusively the relative space utilization of the different types of passenger carriers and the ideal use to which highways could be put when subject to high traffic pressure. Such a test would be an undertaking of large proportions, but when conclusions are based on surveys of actual traffic conditions, they can never completely meet the requirements of a scientific test, and, unless the results are overwhelmingly in favor of one type of carrier against another, must always be subject to contrary interpretations.

METHODS OF RESTRICTING WASTEFUL USE OF STREET SPACE

It is a commonplace of transportation economics that in practice there will not be the most economical utilization of the equipment of the carriers or of the highway facilities provided by the municipality unless the entire transportation service is operated as a unified whole under centralized direction. With competing types of transportation operating under independent management, duplication of traffic facilities on the part of the transportation agencies and consequent wasteful use of highway space are inevitable. It is especially important, therefore, that there be municipal restriction of wasteful use of congested highway facilities where the operation of competitive transportation service tends to weaken the private motives for the elimation of wasteful traffic operations.

The employment of space -utilization tests as the sole basis for the application of traffic restrictions would imply that the value of each unit of passenger transportation service, measured let us say in terms of the carriage of one passenger 1 mile, is uniform, and would leave no room for consideration of possible differences in the classes of persons served by the different types of carriers, in the relative convenience of the different facilities to passengers, or in the rates charged and the operating and other costs incurred per unit of service for the different carriers. It would be necessary, moreover, to avoid adopting any traffic restrictions within the congested area which would seriously disrupt the transportation system of the remainder of the urban area. Under existing conditions, however, the generally hostile attitude of the public toward traffic restrictions provides an adequate safeguard against the too hasty or too severe application of restrictions, and until the public is educated to appreciate the economic injury which results from traffic congestion, it is safe to predict that there will not be as much restriction of traffic as the prevailing conditions justify. But the acceptance by the public of parking restrictions, restrictions on freight traffic in congested areas during business hours, segregation of traffic, and other traffic restrictions which have in recent years been growing rapidly in extent, indicates that if the pressure of congestion becomes severe enough the public will submit in time to the painful necessities of the situation.

It has been suggested that a system of charges for the use of traffic facilities would be the most effective method of restricting traffic to proportions adjusted to existing traffic facilities, and this is the common method whereby the wasteful use of goods and services is restrained. Unless, however, there can be devised a system of charges to which traffic will be subject only as it uses the sections of highways which are congested and only at the periods of congestion, such charges, if heavy enough to exert any influence on the volume of traffic, will operate in the same degree to restrict traffic where there are still unused traffic facilities going to waste as to restrict it where the state of congestion justifies restriction. Except, perhaps, with respect to parking, it does not appear at all likely that any system of charges can be invented and made successfully to operate which will bear heavily on excess traffic while leaving unaffected the traffic for which the facilities are ample.

Traffic charges would tend to repress in greatest degree the marginal traffic, or that traffic which is just worth its cost to those engaged in it, and this is presumably the traffic whose repression would also involve the least economic loss to the community. But there is no assurance that this marginal traffic represents a more substantial proportion of the traffic on congested highways at the periods of congestion than of other traffic and that it is therefore the traffic whose repression would result in the greatest measure of improvement to traffic conditions. . Where restriction of traffic is necessary, it is better to apply it in a flexible manner and in accordance with the needs of traffic rather than arbitrarily by means of traffic charges which fall alike on all users of highway space, regardless of whether that space is congested or not. To suppress traffic which does not contribute to traffic congestion is at least as uneconomic, measure for measure, as to permit traffic congestion to suppress traffic below the traffic capacity of the highways.

EFFICIENCY IN CONCRETE ROAD CONSTRUCTION

A Report of Observations Made on Going Projects by the Division of Control, Bureau of Public Roads

Reported by J. L. HARRISON, Highway Engineer

PART IV.—THE ORGANIZATION AND EQUIPMENT OF A CONCRETE PAVING OPERATION

IN PREVIOUS articles the fact has been developed that under a specification requiring a one-minute mix, 48 batches can be produced by a good mixer every hour. The common causes that prevent full production have been discussed, and there has been some discussion of the methods which have been observed to yield the largest output with the lowest labor and equipment supply. Most of these methods have been subjected to further study on jobs where they were adopted at the suggestion of the representatives of the Bureau of Public Roads in connection with efforts to improve production, and they are, therefore, both by observation and by practical experience with them, known to be efficient methods.

It remains now to show in some detail what equipment is actually needed and what minimum force is required in order to obtain full production if these methods are used, for, though full production is secured, this does not represent full efficiency if it is attained by the use of excessive equipment or labor. However, a statement of methods is not by itself an adequate basis on which to rest a personnel schedule. The operations which must be performed also influence labor requirements. As it is obviously impossible to deal with all of the minor variations in operations which affect the required amount of labor and equipment, only the standard operations will be discussed. But even with this limitation, there is some difficulty of approach to a statement of the personnel and equipment required, for there is no general uniformity in the meaning of even the more standard terms by which construction work is described, and unless the basis of a discussion of this sort is perfectly clear, misunderstanding may easily result.

Thus, in one State, the specifications for concrete paving are interpreted to mean that subgrade (commonly prepared a year or so in advance of paving) is at grade and that the correction of any divergence from this condition shall be paid for as a separate item—excavation. In other States the contractor accepts the subgrade "as is" and understands that the cost of corrections is a part of the cost of paving. The effect on the labor required for "paving" is apparent. In some States shoulder work and the general clean-up are included in the standard unit, "concrete paving," whereas in others shouldering and the clean-up are separate items. In some States a center joint is worked out over the parting strip. In others the pavement is permitted to crack over the parting strip, and in still others no center joint is used. Other items of this sort could be noted if they were needed to illustrate the fact that there are wide differences in the operations required under even so common a term as "concrete paving," as a result of which there are corresponding differences in the personnel and equipment required.

From the standpoint of the engineer, standardization in terminology may perhaps seem to be a matter of little consequence. In general, he feels that no matter how much is included in a given unit of construction, the contractor can make appropriate allowance for it, and that if the units are few the opportunity to claim compensation for extra work will be minimized. A little study of the actual effect of this practice, however, will serve to convince him that some advantages would accrue from standardization. Of these, possibly the greatest would be (1) a broadening of the field of compensation, and (2) the possibility of a more direct comparison of price levels. As matters now stand, the fact that the square yard of concrete pavement may cover all sorts of things not directly associated with its construction and that these will differ from State to State, makes it difficult for the contractor definitely to appraise the situation until he has acquired some familiarity with the correlated requirements and practices locally involved, standards of perfection in vogue, quality of inspection, etc. Thus, as an example, take the matter of requiring contractors to bid on the acceptance of subgrade "as is," as compared with treating the correction of irregular subgrade as a separate item. It is, of course, apparent that variations in the subgrade ordinarily will not be large, but their amount can not be determined except by a rather careful survey. It is also apparent that, because they are commonly small, the unit cost of handling the material which must be moved in correcting them is likely to be high. The contractor who knows local conditions, that is, the one who knows what standards of accuracy are customarily enforced, can bid with some safety on accepting the subgrade "as is," whereas the contractor who is unfamiliar with the local practice would find this a serious element of risk.

In a case observed during the past summer, the existing subgrade, over considerable distances, ran high to the extent of from 6 to 18 inches. On such a subgrade 28 feet wide, reduction to grade might involve moving nearly a cubic yard of material per foot of run—some 5,000 cubic yards to the mile. In this case, it is true that the reduction of the subgrade was paid for as excavation. Cases involving as much extra yardage as this are rare, but it is not uncommon to find subgrades varying in places from as much as 6 inches low to at least that much high, even where the subgrade must be accepted "as is." In such cases there is always a doubt as to what will be required. Some engineers will require that the differences be corrected, some will re-lay the grade line to more nearly fit the existing subgrade. It is not necessary to extend this discussion to justify the conclusion that this lack of uniformity in practice and governing requirements adds to the difficulty which the contractor necessarily faces in any attempt to broaden his field of activity, which is merely another way of saying that lack of standardization in the work covered by common engineering designations reduces competition.

OPERATIONS IN "LAYING CONCRETE PAVEMENT" DEFINED

This discussion will also serve to show that it is necessary to define even so common an expression as "laying concrete pavement" before there can be any certainty that it will mean to the reader what it means to the writer or that the following conclusions as to equipment and personnel requirements, though they are based on extended field observations, will seem to be justified. Therefore the operations covered by the phrase "laying concrete pavement," as herein discussed, are definitely described as follows:

1. *Preparing subgrade.*—This covers all operations incident to converting a rough subgrade, which has previously been brought to the proper grade, to such shape and condition as is required for the placement of the pavement, but does not cover any work required in bringing the rough subgrade to the proper grade.
2. *Handling forms.*—This covers all work incident to taking up used forms, cleaning them, moving, setting, aligning, oiling, and otherwise working with the forms.
3. *Handling materials.*—This covers all work incident to unloading cars, storing or caring for materials, loading job transportation units, and unloading them into the mixer. It also covers the water supply.
4. *Hauling materials.*—All work of transporting materials and all work done on or in connection with the use and care of transportation equipment is included under this designation.
5. *Mixing.*—This includes the operation of the mixer and all work on the mixer.
6. *Finishing.*—All work of placing materials (puddling), finishing, covering, curing, etc., is included under this designation.

Miscellaneous operations, such as the placing of parting strips and reinforcing steel are not standard operations, that is, they are not part of the general operation of laying concrete pavement as it has been treated in these articles, but the effect on the labor requirements of these and other miscellaneous items will be discussed briefly.

It has been noted that the personnel and the equipment for a job depend on the methods in use and the efficiency with which these are carried out, as well as on the operations that are performed. It is therefore necessary at least to outline the methods on which a statement of personnel and equipment rests, just as it has been shown to be necessary to outline the operations covered. The preceding articles have made some reference to methods and such additional reference to them as has seemed necessary is made in presenting the following schedules of personnel and equipment. The methods on which these schedules are based have been selected from among those observed in practice on the various jobs where studies have been conducted, and most of them have been subjected to trial in connection with the efforts made on a number of jobs to increase production. They are the simplest and most direct methods that have been found, and while it is recognized that occasions may arise in which they will prove inappropriate, they at present appear to be the best that have been developed for this work. No job has been found on which all of these methods were in use, but it would appear that there is no basic reason to suppose that in combination they would be less efficient than they have been where observed in use, or where placed in use on recommendation of the bureau's representatives.

COMPLETING THE ROUGH GRADE

Whenever the rough subgrade is not at proper grade the first operation should be to reduce it to grade. The grade, as now staked, is commonly the bottom of the slab at the crown. The "proper grade" is somewhat below this,[1] at the elevation at which the quantity of material which must be taken out for setting the forms and for the thickened edges of the pavement, will be just sufficient to build the crown. If the rough subgrade is consistently high, but by moderate amounts, the blade grader offers the simplest and cheapest means of reducing it to the proper grade. The grader may be pulled by a 5-ton tractor and when so operated will cut away and throw aside excess material expeditiously unless the subgrade material is too rocky or unusually tough. It will not, however, correct a low rough grade. It is, therefore, a more common practice to use a plow and two or three fresnoes on this work, as these will serve equally well to remove high areas and to carry material to low ones. One thing, however, should be emphasized. The current practice of setting forms before the grade is reduced, is cumbersome, and needlessly expensive. It not only requires heavy hand trenching for the forms wherever the grade is high, but also the omission of a section of the forms every 100 feet or so to allow the fresnoes to reach the shoulders and as a result the haul is needlessly extended because all material must be moved through these openings rather than directly to the shoulders. Moreover, the material so handled is commonly wasted in piles, generally on one side of the road only, with the result that it must either be rehandled when the shoulders are worked out or be left to mar the appearance of the roadside.

The force required for this work naturally depends on the amount by which the subgrade varies from the proper grade. A common organization is one plow team and two fresno teams with drivers, a plow holder and a foreman. Where a blade is used the whole width of the subgrade has to be reduced, but when a fresno outfit is used, it is customary to reduce to grade only such a width of the subgrade as is needed for the pavement and for setting the forms. Even then the quantity of material to be handled is often so great that the above organization must be considerably increased, at least for short periods. But since it is advisable to maintain as nearly as possible the same force, these men at least should be regularly employed, and when they are not required on rough grading they can usually be used on shouldering and on the clean-up.

PREPARING THE SUBGRADE

The rough grade having been brought to proper grade (this being in fact a grading operation) the first operation incident to laying the pavement is to cut out for the thickened edge, at the same time cutting wide enough to take out most of the material that must be moved to allow the forms to be set. The proper tool for this work is a fairly heavy blade; and, while this can be drawn by horses, it is better to use a 5-ton crawler-type of tractor as the operating space required is less, and the usefulness of the tractor in general service about the job is greater.

The forms having been set, the width of the subgrade on which the pavement is to rest may be thoroughly broken up by a scarifier and trimmed to correct section by a first-class subgrader, surplus material being removed by hand or by fresno, depending on the accuracy with which the rough grade was finished. In performing this operation, the subgrade should be left from a quarter to a half-inch high and then lightly rolled by a light (1½ to 3 ton) roller after which, to insure accuracy of cross section, the grade of the forms should be checked and the subgrader used again with the blades adjusted to cut to exact cross section. Finally, a modern heavy fine finisher (often known as a subgrade planer) should be attached to the mixer to be dragged along by the mixer whenever it moves If the subgrade is so dry or so solidly packed that it is not easily trimmed by the fine finisher, it should be heavily sprinkled under and about the mixer and whenever necessary, the puddlers should stand on the finisher while it is being moved.

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[1] For a more complete discussion see Efficiency in Concrete Road Construction, Public Roads, January 1926, vol. 6, No. 11, p. 245.

These operations of subgrade preparation require the following equipment:

1 blade grader, heavy-duty, 8-foot blade.
1 caterpillar tractor, 5-ton.
1 scarifier.
1 subgrader.
1 fine finisher.
1 light roller.

The labor requirements are 1 tractor operator, who can act as subforeman; 1 blade operator; 2 laborers with the tractor; 1 roller operator; and 1 laborer at the mixer to take care of the fine finisher and perform miscellaneous tasks, particularly filling depressions in the subgrade, removing cuttings from the fine finisher and wetting the subgrade.

FORM SETTING

Form setting is a manual operation. No heavy equipment is required for it except, as noted above, that the blade grader should take out most of the material which has to be removed before the forms are set. The knack in setting forms is accuracy in preparing the bed on which they are to rest. This bed should preferably be cut a trifle low and topped with a layer of loose material, not over a quarter inch deep, on which the forms rest just high enough so that they must be tamped and worked a little to bring them to exact grade. A proficient laborer can trim out the bed to within a quarter of an inch of the correct level easily and rapidly. If an effort is made to trim exactly to grade, high spots are certain to be left with the result that sections of the form will have to be removed to work them down. On the other hand, if the bed is cut too low—a half inch or more below the correct level—the material under the form must be tamped. This may take the time of one or two extra laborers and there is the added disadvantage that it is seldom so thoroughly done that the forms can be depended on to hold grade.

If the practice of cutting the bed just below grade and setting forms on a thin layer of loose material is followed, 3 men—1 to cut the trench and 2 to set the forms—are all that are required to set and align 1,000 feet of forms a day. Beside this, 2 laborers are required to take up used forms and load them on the wagon in which they are hauled back and distributed to the form setters. A team and wagon is better and cheaper for this work than a truck. It can get over the shoulders and through the ditches better than the truck; and the greater speed of which the latter is capable on good roads is of little advantage in this work because of the time consumed in loading and unloading as well as the bad going. In spite of the fact that it is cheaper to use a team for this work, however, many contractors use a truck because the men do not like to be bothered with the care of a team which, of course, involves extra work after the regular day's work is over. Where there is no other horse-drawn equipment on the job, the best solution is to hire a reliable local man with his team. This relieves the contractor and his foremen of the necessity of looking after a single unit requiring attention after regular working hours.

The work of moving the forms commonly takes less than the full time of the above force: and it is, therefore, available for such other duties as cleaning and oiling the forms, for setting forms when this is necessary, or for any other miscellaneous duties that may be required of them. In addition to the laborers, a foreman is needed to keep things moving smoothly. As the form setting and the subgrading are closely related, it is well to put a good foreman over the two operations, allowing him to use the tractor operator as a subforeman in charge of subgrading operations in his absence.

In this connection a word may be said as to foremen. Some contractors want what they term working foremen—that is, foremen who work with the men doing much the same tasks and assisting generally in the ordinary work of the day. This is a poor practice. Keeping a dozen men effectively at work is task enough to demand the whole attention of any ordinary foreman and if high efficiency is to be attained in pavement construction, it will generally be helpful to make this clear to all foremen. They should understand that they are employed to keep the men under their direction at work and the work for which they are responsible moving smoothly and that, no matter how hard they may work themselves, unless they actually accomplish this, their value is lost. The practices here noted require the following equipment:

1 wagon.
1 team, 2-horse.

The labor requirement is as follows:

1 foreman.
1 laborer to cut trench for forms.
2 laborers to set forms.
2 laborers to remove forms.
1 teamster.

HANDLING MATERIALS

The handling of materials has been discussed in a previous article.[2] Generally speaking, the most satisfactory material plant is an oil or gasoline crane of the crawler type with a ¾-yard bucket (a 1-yard bucket is needed if a high rate of production on the basis of a 6-bag batch is to be had) and a steel bin equipped with a modern fast-dumping, batch-measuring or weighing device. Steam-driven equipment, whether rollers, cranes, or mixers, is out of date and expensive to operate. It commonly requires an extra man to fire each unit, often a team to handle the water supply and sometimes another to deliver the coal; and time is lost in taking on water and often in taking on coal also. The time worked per day is, for this reason, commonly less than can be had from gas equipment and the labor and auxiliary equipment required is commonly more expensive. With most types of heavy equipment there is, to offset these differences, no corresponding increase in production or decrease in any other item of the operating expenses.

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[2] Efficiency in Concrete Road Construction, Public Roads, January, 1926, vol. 6, No. 11, p. 242.

At the cement house, a belt conveyer should be used for elevating the sacks to a point where they can be emptied into hoppers from which the cement can be discharged into the delivery equipment. A sack cleaner and a sack baler should also be installed here, their operation requiring the time of one man. If the cement is sent out sacked, the sacks must be collected and returned to the cement house, a considerable part of the time of one man and a light truck driver often being used on this work. To what extent this is an additional item of expense depends somewhat on whether other conditions surrounding the job are such that a service truck with driver and helper must be maintained in any event. As sending the cement out sacked is not an efficient method of handling it, no provision is made for it in the minimum labor and equipment schedules.

At the mixer the trucks must be turned around before they can be backed to the mixer and dumped. This may be done by backing the trucks but, as on high fills or in deep cuts the room available is limited, it has generally been found more practical to use a turntable. One man is required to operate the turntable and from time to time the tractor is sent back to move it forward out of the way of the mixer.

Turntables are commonly designed to accommodate only one style of trucks. This is regrettable as it often is to the advantage of the contractor to hire a few extra trucks and it is not always possible to procure these, particularly in the heavier types, with a wheel base of the same length as those regularly used. As the practices on any well-organized job become so well established that the initiation of a nonuniform practice in handling a few trucks tends toward disorganization, available equipment is not infrequently rejected, merely because it can not be handled on the turntable. It should be a simple matter to equip the larger sizes of turntables with an easily controlled truck-positioning device which would make it possible to use the turntable in handling trucks of a number of different sizes. If this were done, the contractor's problem in hiring extra transportation to meet extra-long haul would be considerably simplified.

At the mixer, one man is needed to help in dumping the trucks. The end gate must be released to allow the truck to discharge its load and after the load is discharged the gate must be closed and locked. Often a little material not deposited in the skip must be salvaged.

For handling the water, which is, of course, one of the materials, a pump is required. The supply of pipe will depend entirely on local conditions but as a general practice at least 20,000 feet of 3-inch pipe with fittings and a take-out connection for every 300 feet of line should be available. If take-out connections are provided every 300 feet the equipment for water delivery will include two lengths of 2-inch pressure hose about 175 feet long to feed the mixer and, to prevent loss of time in changing hose, the mixer should be provided with a double hose connection so that the second hose can be connected before the first is disconnected.

The pipe is commonly laid before mixing starts and during the move from one mixer set-up to the next. When laid in this way, labor and transportation equipment which otherwise might be idle or assigned to more or less nonproductive work are used. Though pipe must be maintained in position for some days after the concrete is placed in order to furnish water for curing, it generally is possible to synchronize removal and relaying operations with the mixing operation, at least to such an extent that the amount of work to be done on the pipe line when a move is made is materially reduced. In the interest of prompt moving the relaying should be entirely completed before the move is begun. To make this possible the quantity of pipe and fittings which will be required should be calculated before the job is started and the correct quantity of pipe should be sent out. If this is done, removal and relaying can be so handled that the pipe line will never delay the moving; and when handled in this way, the time of two men is all that is required. Generally no extra transportation is needed as the team used for hauling forms can ordinarily find time to move the pipe also.

Finally, to keep the supply of all materials running smoothly it is well to place this work under the direction of a good foreman. It is one of the outstanding important elements of the job and should be carefully supervised.

Handling materials (including water delivery) under these practices, most of which have been more fully discussed in previous articles, will require the following equipment:

1 crane, crawler type, ¾-yard bucket.
1 steel hopper, with modern batch-measuring or weighing device.
1 cement house, with cement loading bins.
1 belt conveyor for cement house.
1 sack cleaner.
1 sack baler.
1 pump, 100-gallon capacity at 400 pounds pressure.
20,000 feet of 3-inch common steel pipe with fittings.
350 feet of 2-inch pressure hose (in two sections).
1 turntable.

The labor required is as follows:

1 plant foreman.
1 craneman.
1 hopper operator.
1 extra man to help unload cars.
3 cement handlers.
1 laborer to clean and bale sacks.
1 pump operator.
2 laborers to handle and lay pipe.
1 turntable operator.
1 truck dumper.

MATERIAL DELIVERY

The equipment required for material delivery and transportation has been fully discussed in another article.[3] As this is the most variable element on a paving job, it must be determined for each project. Besides the transportation equipment each job should have a reasonably well-equipped repair shop with a skilled mechanic in charge to keep the equipment in good running order. Naturally, as such a man may be expected to spend most of his time working on the trucks, he should be particularly well trained in caring for these, but he should know something of other types of equipment as well. On most jobs the mechanic should have a helper who may well know something of blacksmithing.

There are a few key positions on any sort of work, where it is outstandingly expensive to hire cheap men.

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[3] Efficiency in Concrete Road Construction, PUBLIC ROADS, December, 1925, vol 6, No. 10, p. 220.

The job mechanic holds one of these positions. The craneman holds another, the mixer operator another. In any of these positions the best man that can be obtained is not too good. As high production always means extra-hard work for these men, a wage scale based at least in part on the production obtained is always worthy of consideration. A scale based wholly on production would from the contractor's standpoint, be perhaps even more desirable, but the objection to this lies in the fact that superintendence quite generally is the weakest element in the contractor's organization, and these men are not in a position to develop high production if the superintendence is weak. They can only respond wholeheartedly when good superintendence makes high production possible. In view of the common occurrence of poor superintendence, it is unfair to ask such men to work wholly on a production basis for the risk they then assume is too great. A far better and fairer way is to pay the standard wage and offer a bonus for production over some specified quantity.

EQUIPMENT AND LABOR REQUIRED IN MIXING

Efficiency in mixing requires a number of things. The paver should have ample power. It should be able while mixing to move and at the same time drag a heavy fine finisher, and also to start the skip and the discharge as close together as the operator can move the levers. Moreover, the real test of a mixer is not whether it can do these things when new but whether it can do them after it has been in service two or three seasons. This requires a good power plant. The mixer should be able to move any reasonable distance, 10 miles or more, under its own power without stopping to cool bearings or clutches. This is essential as a tractor is not always at hand to drag it from one location on the job to another.

Too much care can not be taken by a contractor who is buying a new mixer in determining what time is required in emptying the skip after it is in a vertical position. A good 5-bag paver will ordinarily clear the skip in less than 5 seconds, yet one recently observed by the writer took over 30 seconds to complete this operation. Poor blade design seemed to be at the bottom of the trouble in this case, and the slow charging and discharging cut production from a possible 48 to about 34 batches an hour. No contractor can afford to own such a mixer. The time required to start the discharge after the time bell rings should be determined and the time required for discharging should be examined. It should be remembered, also, that there is an appreciable difference between the time required to discharge a batch of sloppy concrete and that required to discharge a batch of the consistency now commonly used in highway work. The latter runs more slowly in the chute, piles up in the generally inadequate bucket, and finally backs up into the chute with the result that the discharge time is increased. The rate of discharge and the rate at which the skip is raised depend somewhat on the speed at which the engine is running. What the buyer wishes to know is how fast these operations take place under normal conditions and for that reason he should be sure that the drum speed is normal—about 15 revolutions a minute—and that concrete of standard consistency is being mixed when checking performance in these respects.

Aside from the mechanical operation of the mixer, which is, of course, vital to high production, purchasers of new machines would do well to give thought to the quality and workability of the concrete produced. In this particular there is, perhaps, as much difference as there is in mechanical efficiency. Smooth, well-mixed concrete is easier to work than brash, undermixed concrete. Engineers now give little thought to any phase of this matter except the slump. But a given slump may be had in either of two ways: (1) By thoroughly mixing a proper volume of water with a given amount of cement and aggregate; and (2) by undermixing the same aggregate and cement with excess water. If this latter is the case the resulting concrete is generally admitted to be of relatively poor quality. That the use of excessive water accompanied by undermixing is a frequent result of current specifications is evidenced by the large number of jobs on which the typical result of this practice may be observed. The result referred to is the rising of water to the surface of the newly-laid concrete in such quantities that it drips over the forms often for an hour or so after the concrete is placed. Where the water content is correct and the mixing is well done, no water will drain over the edge of the forms. If the contractor feels that this is the engineer's business, it may be well to suggest that his self-interest should dictate the selection of a really good mixer for the very good reason that a well mixed concrete is easier to handle and finish than an undermixed concrete.

The time is coming, and that at no distant date, when the present ineffective treatment of the mixing problem will be replaced by some practical and direct method of insuring thorough mixing and a correct water content. There are, then, a number of reasons that should influence contractors to protect themselves by purchasing only those mixers which, within the specified time, will turn out a really well-mixed batch. There are such mixers and they can be readily distinguished by observing the concrete mixed in them for one minute to a slump of an inch or less (1) as to the ease with which it can be finished and (2) whether water rises to the surface and runs off over the forms. If the mixing has been well done, the finishing will be relatively easy, the surface will require no final correction and no water will run off over the forms.

The equipment requirement for mixing is one mixer and the labor requirement is one operator.

FINISHING EQUIPMENT AND PERSONNEL

The first operation in connection with finishing is the spreading of the concrete—commonly called puddling. If the mixer operator is really well trained he will spread the mixed concrete so well that two puddlers will be able to do all the spreading necessary and do the spading along the forms as well. No heavy equipment is needed in connection with this operation.

The next operation is shaping the crown and working and compacting the mass, which is now almost entirely done by machine finishers. A sliding finisher using two sliding screeds does this work outstandingly well. When operated over concrete of proper and uniform consistency, the surface produced is conspicuously uniform. Following immediately behind this machine one man with a long-handled float can readily wipe out the screed marks, and check the surface with a 10-foot straightedge. The finish obtained in this way is rapid and so accurate that it is seldom necessary to fill a low spot or to take off a high spot in order to meet the most rigid requirements now in force as to surface finish. The final belting follows the checking of the surface with the straightedge, and, for this operation, the finishing-machine operator ordinarily can be called on to aid the finisher as his time is seldom taken up fully in running the finishing machine. Covering the pavement with burlap or other temporary cover commonly requires two men; the earth covering commonly requires two more; and watering another.

The equipment required for finishing is, then:

1 finishing machine, double sliding screed.

The labor requirement is:

2 puddlers.
1 finishing-machine operator.
1 finisher.
2 laborers to spread canvas.
2 laborers to cover concrete with earth.
1 laborer to sprinkle the concrete.

In addition to the labor requirements above discussed, common practice and good judgment dictate the employment of a superintendent, a timekeeper, a watchman, and a water boy. In the consideration of the labor and equipment required in laying the pavement, however, these are not included because their duties extend to the whole job, and they belong, therefore, to the job overhead rather than to the labor structure of any unit of the job.

MISCELLANEOUS LABOR AND EQUIPMENT REQUIREMENTS

A center joint is now quite commonly required. Where it is used, a line of reinforcing steel is often placed along each edge. Sometimes other reinforcing is also used. Placing the center joint and the two lines of reinforcing steel generally requires the time of

TABLE 1.—*Major equipment used on typical concrete paving jobs during 1925 compared with the most efficient equipment combination*

Operation	Most efficient equipment combination	Equipment combinations on typical jobs																
		A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q
Rough grading:																		
Plow	1			1		1									1			2
Fresno	2			1														3
Wheel scraper									1									
Mancy grader												3						
Truck															1			
Fine grading:																		
Blade grader	1	1		1	1	1	1	1	1	1		1	1	[1]1	2	1	2	1
Tractor	1	1	1	1	1	1	1	1	1	1	1	1			2	1	1	1
Scarifier	1				1							1			1	1		
Subgrader	1	1	1	1		1	1	1		1	1				1		1	1
Fine finisher	1	1			1							1						
Roller	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Fresno, team		2	1	1	2	2						[2]4			1	1	2	2
Wagon		1															2	
Drag					1													1
Handling forms:																		
Forms, lineal feet	5,000	4,000	2,800	3,200	2,500	2,100	2,500	5,000	2,000	4,000	3,000	3,000	3,000	[3]1,000	3,200	3,000	3,600	4,000
Wagon, team	1	1	1	1	1		1	1	1	1	1	1	1	1	1	1	1	1
Handling materials:																		
Crane	1	1	1	[4]1	1	1	1	1		1	[4]1	2	2		1	[4]1	1	1
Hoppers	1	1	2	1	1	1	1	1		1	2	2	1		1	1	1	1
Conveyor (for cement)	1						1											
Cement house	1	1	1	1		1	1	1	1	1	1	1		1	1	1	2	2
Pump	1	2		1	1	1	1	1	1		1	1			2	2	2	
Pipe, lineal feet—																		
3-inch	20,000																	
2½-inch				15,000		13,200												
2-inch		17,000	15,000	12,000	26,400		15,840	18,480		1,000	15,840	31,680	10,560		15,840	42,240	52,580	20,000
1½-inch			10,000															
Size unknown									15,840									
Hose, lineal feet—																		
2-inch	350	125	200							150	150	150		300		300	200	
1½-inch				150	200	200	400	250	([5])				200		150			270
1-inch			200	150													450	
¾-inch																400		
Size unknown																		200
Turntable	1	1	1		1	1	1	1	1						1	1		
Sack cleaner	1	1		1		1	1	1	1									
Sack baler	1			1		1	1	1	1									1
Loader									2					1				
Stacker									2									
Hauling materials:																		
By truck—																		
Trucks	20	20	16	[6]8	11	12	[6]8	[6]6	26	[7]12	[8]13	21	[9]6	[6]4	40	26		
Tractor							1											
Service truck	1	4		1	1								1		1	2		
By industrial railway—																		
Locomotives																	[10]18	[11]3
Cars																	227	52
Batch boxes																	239	104
Track, miles																	[12]14	[13]6
Switches																	15	5
Service trucks																	1	1
Mixing:																		
Mixer	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
Mixer crane																	1	1
Finishing:																		
Finishing machine	1	1	1	1	1	1	1	1			1	1	1		1	1	1	1
Burlap truck				1														
Miscellaneous:																		
Repair shop	1	1		1	1											1		
Straw wagon					1								1		1	1		
Lighting plant							1										1	
Water truck				1														

[1] Horse-drawn.
[2] Slip scrapers.
[3] Used at center only; curb at side.
[4] Steam crane.
[5] Length unknown.
[6] 2-batch trucks.
[7] Nine 1-batch and three 2-batch trucks.
[8] 3-batch trucks.
[9] 3 and 4 batch trucks.
[10] Ten 8-ton, one 6-ton, and seven 5-ton gas locomotives.
[11] 8-ton locomotives.
[12] 7 miles of 25-pound and 7 miles of 20-pound track.
[13] 25-pound track.

two men. If the edges are rounded and a center joint is formed over the parting strip, one or two extra men may be required. Working the center joint, however, is of no apparent value as the concrete cracks by contraction over the parting strip within a few hours after it is placed and the black center line, now widely used to divide traffic, fills and obliterates it. Where the worked joint is used, the final result is about the same, except that, on the whole, the center band put down over the crack ordinarily presents the neater appearance.

The common tendency of pavements to develop transverse cracks together with the fact, increasingly to be observed as pavements grow older, that where no expansion joints are used, the buckling which occurs during the summer months tends constantly to increase the roughness of these pavements, leads to the observation that transverse joints are not now used to the extent that they should be. At one time they were in common use, but they fell into disuse because they were expensive to install and because it was difficult to obtain a uniform finish across them. The sliding finisher has eliminated at least the more important part of this difficulty. It is now possible to finish over such a joint without the slightest difficulty and with full

TABLE 2.—*Personnel employed on typical concrete paving jobs during 1925 compared with personnel required with most efficient equipment*

Operation	Personnel required with most efficient equipment combination	Personnel employed on typical jobs																
		A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q
Rough grading:																		
Grade foreman	1		1															
Plowholder	1																	
Teamsters	2			2					1			5						1
Laborers									1									
Fine grading:																		
Subcontractor									1									
Foreman		1	1	1	1	1	1	1	1	1	1	1			1	2	1	1
Tractor operator	1	1	1	1	1	1	1	1			1	1			2	1	1	1
Blade operator	1				1	1	1	1	1			1	1	1	2	1	1	1
Roller operator	1	1	1		1	1	1	1	1	1	1	1	1	1		1	1	1
Subgrader operator	1																	
Teamster		2		1	2	2		1								1	1	2
Laborer	2	9	13	5	10	13	11	13	7	1	18		12	7	11	43	7	10
Handling forms:																		
Foreman	1	1	1	1		1	1	1	1		1	1				1	1	
Teamster	1	1	1	1	2	1	2	1	1	1				1	1	1		
Laborer, to set forms	3	2		1	2	2			2	2	3	2	5	2	2	5	2	2
Laborer, to strip forms	2	2			2	2			2	2	3	2	5	2	2		2	2
Laborer			4	3	2	1	3	4		2		2		2	1	1	2	2
Handling materials:																		
Plant foreman	1	1		1		1	1	1	2		1	1	1					
Crane operator	1	1	1	1	1	1	1	1		1	1	3	1		1	1	1	1
Crane fireman				1			1	1			1					1		
Hopper operator	1	1	1	1	1	1	1	1		5	2	2	3	4	2	2	2	2
Cement handlers	3	5	1	5	3	6	3		5	3	4	3	2	1	2	2	5	6
Bag dumper			3			2	2	3	3	3	4				2	2		
Bag baler	1		1	1		1	1	1	1									1
Pump operator	1	1		1	1	1	1	1	1		1	1			1	1	2	1
Laborer on pipeline	2	1					2				1	1			1	1	1	1
Turntable operator	1	1	1		1	1	1	1	1									
Truck dumper	1	1		1	1	2	1	1	1	2			1	1	1	1		
Laborer	1	1	2	3	1	1		14	1			2	5	6	1	2	1	5
Batch box handlers																	2	3
Hauling materials:																		
By truck—																		
Foreman		1										1						
Mechanic	1	1	1	2	1		1		1							1		
Helper	1	1		1														
Drivers, delivering material	[1] 12	20	11	[2] 8	11	12	[2] 7	[2] 6	16	12	[3] 11	16	[4] 5	[2] 4	40	36		
Truck checker																1		
Service truck driver		2		1	2								2		3	3		
By industrial railway—																		
Foreman																	1	1
Locomotive engineers																	10	3
Brakemen																		3
Switchmen																	1	
Mechanic																	2	1
Mechanic's helper																	2	
Laborer																	2	
Truckmen																	3	10
Tractor operator																	1	
Service truck drivers																	5	1
Mixing:																		
Foreman		1										1			1	1	1	
Mixer operator	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	2
Mixer crane operator																		1
Finishing:																		
Foreman				1		1		1										1
Puddler	2	3	3	2	3	2	2		3	4	4	2	3	4	2	3	3	4
Finishing machine operator	1	1	1	1	1	1	1	1	1		1	1	1		1	1	2	1
Laborer, to finish concrete	1	3	4	2	3	6	2	3	2	2	2	2	3	3	3	4	2	2
Laborer, to spread canvas	2		2	2	1		2				3		2			2		2
Laborer, to cover concrete	2				2		2										3	
Laborer, to sprinkle concrete	1	2	2			5	6	1	4		3						2	
Laborer, to spade concrete				1	1									2	2	2		
Laborer		1		4					2	4		3	4		1	1	1	
Miscellaneous:																		
Superintendent	1			1		1	1	1	1			1			1		1	1
General foreman					1			1	1		1		2	4	1			
Timekeeper	1			2		1	1	1	1			1		1	1		1	1
Watchmen	1	1	1	2	2	2		1	1	1			2	1	1		1	1
Waterboy	1			1			2	1							1	1		1
Steel setter			1							3	6		1	2				
Joint material setter		1	1		1	1	1	1				1			1	1	1	4
Total (without drivers or miscellaneous)	42	52	47	49	48	58	53	56	48	35	57	40	53	38	47	90	61	71

[1] Average number for season.

[3] 3-batch trucks.

[2] 2-batch trucks.

[4] 3 and 4 batch trucks.

assurance that high joints are wholly unnecessary. Moreover the use of joints need not at all reduce the rate of pavement production. An extra man, sometimes two extra men, will be required if expansion joints are used and some extra time is required in finishing, because the joints must be cleaned off carefully on top and along the ends, but as no other difficulties need now attend their use and as pavements where they are used appear to maintain their smoothness better than the jointless pavements, it is probable that their use will become more general.

The labor and equipment requirements here scheduled are believed to represent the minimum for full production. They are shown in relation to the labor and equipment used on a number of the actual jobs studied by the bureau in Tables 1 and 2; and a comparison of the minimum requirements as presented in this article (column 1 of both tables) with the jobs listed will suggest that better superintendence can reduce the labor and equipment used.

For the more perfect understanding of the labor table it may be well to remark that on a large percentage of the jobs on which statistics have been taken, the methods in use so intimately associated rough grading and fine grading that a definite separation was impossible. It was often found, too, that a good deal of the labor was hired to perform general operations as directed; that is, it was not constantly assigned to a specific task. For purposes of comparison, the totals shown omit truck drivers hauling material—an item that should vary with the length of haul—and miscellaneous positions such as the superintendent, timekeeper, water boy, etc. By this means a more direct comparison of the labor used on the operations discussed above is possible. It will, of course, be noted, that the major variation is in the grading. This is unavoidable.

One other matter deserves mention here, namely, that in many of the operations incident to laying a concrete pavement the labor requirement is fixed; that is, it can not be reduced even though the rate of production falls off. Thus, at the material plant, a crane-man, a helper to assist in unloading cars, a hopper operator, and three men handling cement appear to be an absolute minimum. The equipment can not consistently be operated with less than this force even if production falls to 100 feet a day. On the other hand, some members of this force are not working to capacity even when the 5-bag paver is running at 48 batches an hour. Much the same is true of most of the other operations. The subgrade crew, for instance, requires a tractor operator, a roller operator, and at least two men, no matter how low production falls, because the equipment can not be operated with less. The mixer can not be operated by less than one man and the finishing can hardly be reduced below a man on the finishing machine and one on the float. There is no need to continue the recital of these minimum requirements to confirm the statement made in other articles of this series that cutting down the number of batches per hour (as is done whenever the mixing time is extended to 1¼, 1½, or 2 minutes) raises the cost of paving because neither the investment in equipment nor the amount of labor employed can be correspondingly reduced. Rather the path toward lower cost lies along the line either of increasing the size of the mixer or of reducing the time of mixing. By either process a closer approach can be made to the full utilization of the capacity of the men and equipment used in the secondary operations.

"The men don't seem to be working a bit harder," remarked a contractor when production on his job had been raised from an average of about 63 feet an hour to an average rate of nearly 100 feet an hour. That was visibly true. The difference was simply in the fact that they were wasting less time. How much further this would have been true remains to be seen. It is, however, apparent that a high-grade mixer running at the rate of about 15 revolutions a minute can turn out first-class concrete in a minute. Experiments are under way to determine whether, if the speed is run up to about 20 revolutions per minute, the mixing time can be reduced proportionately and still produce concrete of equal merit. If so, the mixer cycle might be reduced to about 60 seconds with the result that 60 batches an hour would represent full efficiency for a 5-bag mixer. The studies which have been made suggest that it probably would be possible for the correlated equipment (except the crane) to meet this rate of production with almost no change in the amount of labor employed. Further study, however, is required before any final conclusions as to this matter can be drawn.

BOND BETWEEN CONCRETE AND STEEL

"Studies of Bond between Concrete and Steel," by Duff A. Abrams, has just been published as Bulletin 17 of the Structural Materials Research Laboratory, Lewis Institute, Chicago. Bond tests were made by applying a pull on one end of 1-inch plain round steel bars embedded axially in 8 by 8 inch concrete cylinders; parallel compression tests were made on 6 by 12 inch concrete cylinders. The concrete covered a wide range in quantity of mixing water, cement, and size and grading of aggregate. Tests were made at ages of 7 days to 1 year; 735 pull-out bond tests and 735 parallel compression tests were made.

The principal conclusions from the tests are:

(1) Slipping of the bar began at a bond stress of about 10 to 15 per cent of the compressive strength of the concrete, but considerable additional load was taken before the ultimate bond resistance was reached.

(2) An end slip of bar of 0.0005 inch occurred at 55 to 60 per cent of the maximum bond. For mixtures leaner than 1 : 1, the maximum bond was about 24 per cent of the compressive strength of the concrete and came at an end slip of about 0.01 inch regardless of the characteristics of the concrete.

(3) Bond and compressive strength increased with age of the concrete from 7 days to 1 year. For 1 : 5 concrete of water-ratio 0.88, the bond at 1 year was 134 per cent of the 28-day value and the compressive strength was 148 per cent.

(4) Bond responded to changes in water-ratio of the concrete in much the same way as compressive strength; increase in water-ratio due to use of wetter concrete, less cement, or an excess of fine aggregate, resulted in material reductions in both bond and compressive strength.

(5) For mixtures richer than 1 : 1 the bond fell off probably due to the greater volume changes during hardening, which is characteristic of such mixtures.

(6) The use of 4 per cent of the 28-day compressive strength of concrete as the working stress in bond for plain bars, as specified by the joint committee, is justified; this gives a factor of safety of about 2½ to 3 against first slip.

(7) The use of crude oil to replace mixing water, in general, caused a reduction in both bond and compressive strength of concrete due probably to the lubricating effect of the oil. Replacing cement with hydrated lime also decreased the compressive strength and bond about 1.2 per cent for each 1 per cent of hydrated lime in terms of volume of cement or about 2 per cent for each 1 per cent by weight.

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ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

No. 105. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136. Highway Bonds. 20c.
220. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314. Methods for the Examination of Bituminous Road Materials. 10c.
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387. Public Road Mileage and Revenues in the Southern States, 1914.
388. Public Road Mileage and Revenues in the New England States, 1914.
390. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
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No. 1259. Standard Specifications for Steel Highway Bridges; adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULAR

No. 94. TNT as a Blasting Explosive.
331. Standard Specifications for Corrugated Metal Pipe Culverts.

FARMERS' BULLETINS

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*849. Roads. 5c.
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OFFICE OF THE SECRETARY CIRCULARS

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59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying Out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables Upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 20, D- 4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

JANUARY 31, 1926

STATES	FISCAL YEARS 1917-1925			FISCAL YEAR 1926									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1925			PROJECTS COMPLETED SINCE JUNE 30, 1925			* PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 5,970,097.71	$ 2,863,197.86	611.8	$ 8,315,672.36	$ 3,970,792.98	473.7	$ 7,310,919.97	$ 3,650,765.85	355.6	$ 1,086,634.09	$ 498,492.42	28.2	$ 3,466,205.89	Alabama
Arizona	9,580,133.43	5,016,119.94	613.8	1,041,126.35	636,703.27	92.2	1,217,174.81	805,183.26	88.7	176,977.10	108,150.70	26.7	3,051,091.83	Arizona
Arkansas	13,310,190.08	5,380,181.73	1048.9	3,508,401.67	1,540,533.11	189.5	5,937,644.37	2,851,698.28	371.6	653,819.17	294,714.62	44.9	1,538,676.26	Arkansas
California	22,346,175.99	10,719,249.61	894.8	2,664,358.92	1,206,974.63	72.4	11,889,525.68	6,873,052.11	356.0	341,396.01	133,340.26	6.3	4,140,198.39	California
Colorado	11,876,703.94	6,067,814.34	651.2	1,029,861.87	530,202.26	41.7	4,369,258.70	2,193,804.29	234.1	464,225.60	260,348.34	25.2	3,273,642.77	Colorado
Connecticut	4,558,639.29	1,819,368.66	101.6	750,965.91	250,212.14	13.4	1,901,818.16	590,259.68	29.1	238,929.61	62,777.95	3.7	1,611,062.57	Connecticut
Delaware	4,281,569.81	1,496,190.65	107.1	455,744.13	212,544.95	12.3	980,042.66	388,784.75	22.7	445,659.65	9,000.00	10.5	367,537.65	Delaware
Florida	2,959,273.72	1,405,487.97	36.3				11,039,134.35	5,071,300.29	295.5	635,098.94	207,255.00	13.8	1,400,910.74	Florida
Georgia	20,156,002.37	9,406,366.46	1478.3	2,979,105.82	1,447,535.03	181.5	11,103,688.50	6,240,139.13	691.7	927,564.99	459,146.55	71.5	1,878,765.83	Georgia
Idaho	9,394,676.80	4,815,332.26	600.1	735,647.37	484,367.73	47.1	2,651,708.09	1,633,247.23	183.4	471,051.34	255,404.45	17.0	1,371,275.33	Idaho
Illinois	40,010,481.10	18,640,076.28	1236.2	933,608.56	447,871.39	32.1	8,398,091.85	4,076,691.59	285.3				6,667,558.74	Illinois
Indiana	13,639,172.65	6,562,455.68	422.1	1,195,563.59	573,819.54	47.0	17,936,040.96	8,415,742.71	487.0	543,575.62	263,346.52	16.6	2,382,990.55	Indiana
Iowa	27,272,285.21	11,107,492.99	1996.9	722,462.52	320,365.06	36.5	7,838,501.88	3,526,869.59	476.8	1,184,561.72	572,403.90	140.3	3,958,431.46	Iowa
Kansas	26,399,695.77	9,755,273.32	831.4	3,949,959.30	1,760,737.02	233.9	10,389,649.37	4,283,091.36	500.3	2,831,782.29	710,832.29	152.5	2,954,477.01	Kansas
Kentucky	14,832,324.28	6,205,994.59	584.9	3,194,286.55	1,218,841.98	84.6	7,435,186.66	3,400,478.36	313.6	272,730.33	119,468.16	9.0	2,268,025.91	Kentucky
Louisiana	11,939,424.97	5,279,870.86	927.6	1,079,810.60	503,534.86	68.9	3,283,966.29	1,591,576.56	160.4	617,313.13	308,656.55	25.5	1,588,769.17	Louisiana
Maine	8,174,281.31	3,907,870.33	281.4	249,291.85	123,602.05	9.8	2,046,840.72	825,486.82	66.2	188,780.85	94,390.42	8.3	1,513,478.38	Maine
Maryland	8,132,506.90	3,649,383.15	294.4	1,290,968.13	576,326.77	41.8	1,840,919.16	838,622.00	99.2				660,725.08	Maryland
Massachusetts	14,047,656.22	5,467,661.28	300.6	751,829.45	271,111.11	20.4	6,087,949.31	1,591,752.67	80.4	979,762.40	244,140.00	14.6	2,534,060.94	Massachusetts
Michigan	16,234,000.80	7,328,316.91	612.6	7,172,875.22	3,284,579.34	272.1	10,792,830.57	4,978,793.08	281.6	501,034.00	208,769.57	11.9	4,541,906.10	Michigan
Minnesota	30,415,685.89	12,738,642.04	2721.2	5,271,152.99	2,284,974.52	397.0	7,688,721.98	2,433,300.00	522.4	669,056.29	123,000.00	37.9	2,011,863.44	Minnesota
Mississippi	10,292,265.79	4,986,702.73	803.4	1,970,556.59	984,316.58	156.9	8,680,615.97	4,251,609.44	427.8	607,557.96	303,778.96	65.0	1,599,610.30	Mississippi
Missouri	17,368,156.57	8,219,411.43	1118.9	4,262,723.91	2,066,541.56	190.3	25,054,337.23	9,863,232.64	698.6	1,095,747.26	31,375.43	29.1	2,615,874.94	Missouri
Montana	10,156,600.41	5,317,523.15	921.6	705,847.87	528,905.67	71.2	1,779,748.42	1,306,965.25	175.2	1,016,469.79	565,015.94	119.0	5,706,474.99	Montana
Nebraska	9,306,374.36	4,389,523.50	1570.6	1,521,442.87	736,793.25	120.3	10,923,742.34	5,410,993.94	1079.6	435,168.68	216,080.08	40.7	3,881,844.23	Nebraska
Nevada	4,917,465.69	3,088,299.78	357.3	1,363,562.91	1,146,956.53	125.6	4,203,299.63	3,507,647.17	367.9				1,053,311.52	Nevada
New Hampshire	4,165,687.86	1,986,226.87	208.1	766,008.74	361,274.93	27.6	648,566.19	304,202.46	19.5	9,979.13	4,989.56	0.5	512,798.18	New Hampshire
New Jersey	11,961,357.45	3,820,679.99	219.1	2,112,044.24	581,403.46	35.1	8,463,105.76	3,079,655.72	56.4				985,680.83	New Jersey
New Mexico	8,717,999.18	4,914,070.61	1081.3	3,545,064.81	2,313,941.49	329.6	1,476,466.48	972,341.77	116.3	74,555.37	46,757.90	5.4	2,725,274.23	New Mexico
New York	28,597,769.67	12,229,076.53	831.5	7,008,005.43	2,846,740.06	175.5	32,436,625.75	9,808,265.30	621.7	9,053,570.00	2,222,840.00	148.6	6,938,273.11	New York
North Carolina	21,014,450.41	8,746,454.59	1119.8	2,964,398.83	1,243,624.94	79.6	8,120,203.86	3,414,734.91	190.1	1,790,503.65	597,253.57	35.0	1,715,137.99	North Carolina
North Dakota	10,829,263.82	5,268,930.47	1917.5	778,568.75	388,974.47	164.5	4,282,459.87	2,143,144.39	533.0	926,305.53	463,152.66	131.0	2,484,457.01	North Dakota
Ohio	41,572,252.81	15,244,993.93	1191.1	4,626,351.05	1,574,690.48	129.7	9,817,064.82	3,843,082.39	306.3	2,077,246.26	669,907.97	57.5	4,399,121.23	Ohio
Oklahoma	20,787,024.94	9,672,890.34	852.2	2,570,086.53	1,194,939.18	112.3	6,364,192.65	3,012,797.46	291.0	719,754.38	227,746.86	20.2	1,951,413.17	Oklahoma
Oregon	14,388,188.70	7,142,364.63	794.6	1,409,879.93	767,336.36	75.3	2,817,306.00	1,644,702.27	144.2	1,536,179.15	118,576.27	42.8	1,206,367.47	Oregon
Pennsylvania	43,054,836.19	16,222,023.97	850.3	4,442,508.99	1,319,739.45	80.9	35,951,160.10	9,744,058.97	663.7	1,829,834.89	600,285.62	44.7	3,452,672.99	Pennsylvania
Rhode Island	2,628,496.20	1,119,688.09	64.8	481,300.30	154,988.63	6.0	2,380,016.40	654,003.63	40.2	263,012.75	63,135.00	4.2	675,753.65	Rhode Island
South Carolina	11,163,347.84	5,121,267.64	1235.9	1,161,737.54	412,561.25	58.6	6,014,054.74	2,827,005.77	345.2	1,913,660.91	579,602.92	78.1	861,086.52	South Carolina
South Dakota	12,091,434.67	5,989,879.00	1447.9	3,089,616.22	1,480,829.11	387.6	5,037,902.24	2,426,523.01	764.4				1,269,558.88	South Dakota
Tennessee	13,789,140.98	6,732,079.77	497.9	3,119,078.93	1,516,196.20	122.7	10,018,067.48	4,472,948.74	316.8	1,318,895.56	621,612.00	58.7	1,937,754.29	Tennessee
Texas	54,120,970.83	21,057,940.12	3907.1	8,890,315.94	3,677,963.16	540.5	20,102,943.89	8,885,378.47	1168.4	4,827,819.52	2,198,083.57	248.5	4,787,065.68	Texas
Utah	6,259,159.41	3,816,836.91	423.1	448,533.65	294,374.47	10.5	2,922,204.30	2,014,512.26	251.9	279,557.40	212,710.59	16.0	1,478,344.77	Utah
Vermont	3,015,174.51	1,452,894.45	107.8	200,295.47	100,082.03	3.5	1,983,964.60	865,348.64	46.2	59,192.10	29,596.05	0.1	800,585.83	Vermont
Virginia	13,099,720.01	6,271,998.20	676.2	6,494,805.76	3,008,514.78	241.3	5,980,342.65	2,623,564.04	195.2	2,177,236.66	971,726.94	52.1	525,710.04	Virginia
Washington	13,362,504.18	6,117,211.87	526.7	3,392,361.37	1,510,538.36	137.0	1,982,719.23	963,400.00	26.7	757,215.96	261,000.00	1.2	1,293,625.77	Washington
West Virginia	7,343,200.86	3,230,293.33	326.7	252,664.30	123,013.89	12.8	7,002,615.99	2,787,834.00	185.7	327,937.93	133,956.79	11.3	1,077,412.99	West Virginia
Wisconsin	21,807,140.91	8,919,640.62	1451.7	1,106,354.69	547,215.65	54.5	5,552,066.62	2,728,635.42	272.1	343,475.86	171,345.00	19.2	5,071,978.31	Wisconsin
Wyoming	8,609,819.33	4,739,096.67	982.0	1,494,137.50	907,603.81	102.8	2,999,872.84	1,877,948.36	208.8	28,580.20	18,348.00	0.1	1,023,277.16	Wyoming
Hawaii							342,277.22	97,440.00	6.5				1,002,713.00	Hawaii
TOTALS	740,140,790.82	325,654,346.00	41898.3	117,469,945.28	53,434,689.49	5920.1	375,377,466.20	169,702,616.03	15421.0	46,639,239.02	16,268,515.36	1893.4	116,314,833.12	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 117,419,095.53 Federal aid $ 50,407,812.60 Miles 4583.3

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